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Load Rating of the Lahontan Arch Spillway Bridge at Fallon, NV

Henry Díaz-Álvarez and Orlando Carrasquillo-Franco

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Final report

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Abstract: This study focuses on the load-rating analysis of a five-span, concrete arch bridge located at Fallon, NV. The Lahontan Dam Spillway Bridge was constructed in 1915 and, after approximately 94 years of service, the bridge has deteriorated. The U.S. Army Engineer District, Sacramento, inspected the bridge in 2009 and determined that the bridge is in poor condition. Advanced section loss, deterioration at different locations along the full length of the bridge, and severe spalling in the three main arch beams and in the piers were the main defects observed. These defects put in jeopardy the structural integrity of the bridge; thus, the bridge is currently closed to traffic. The purpose of the load rating was to identify the safe load capacity of the bridge under today's design vehicle and legal loads. The load ratings were performed with two specifications, the American Association of State Highway and Transportation Officials' (AASHTO) *Manual for the Condition Evaluation of Bridges* load factor rating and the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*. The load ratings were performed for the following types of loading: notional HS20-44; the HL93; and the legal load for vehicles Type 3, Type 3S2, and Type 3-3. The rating factors' results confirmed the poor condition of the bridge described in the 2009 inspection report. It was concluded that the bridge design does not follow current standards; thus, a weight limit sign is required to be posted on the bridge. The recommended weight limit based on the LFR at inventory level is 7 tons of the Type 3 loading, 11 tons of the Type 3S2 loading, and 14 tons of the Type 3-3 loading.

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Preface

This report describes the load-rating process conducted for the Lahontan Spillway Bridge, Fallon, NV. This project was coordinated by Henry Díaz-Álvarez of the U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS.

This investigation was sponsored by the U.S. Army Engineer District, Sacramento. This report was prepared by Henry Díaz-Álvarez and Orlando Carrasquillo-Franco under the supervision of Terry R. Stanton, Chief, Structural Engineering Branch; Bartley P. Durst, Chief, Geosciences and Structures Division; Dr. William P. Grogan, Deputy Director, Geotechnical and Structures Laboratory (GSL); and Dr. David W. Pittman, Director, GSL.

COL Gary E. Johnston was Commander and Executive Director of ERDC. Dr. Jeffery P. Holland was Director.

Unit Conversion Factors

Multiply	By	To Obtain
feet	0.3048	meters
inches	0.0254	meters
inch-pounds (force)	0.1129848	newton meters
kips (force)	4.448222	kilonewtons
pounds (force) per square inch	6.894757	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

The calculations summarized in this report provide a bridge load rating based on the inspection conducted by the U.S. Army Engineer District (USAED), Sacramento, in 2009. A detailed comparison of load-rating procedures and vehicles is also presented as a guide to the U.S. Army, as it refines its load-rating procedures.

The load ratings are performed with two specifications, the *AASHTO Manual for the Condition Evaluation of Bridges* **(MCEB)** and the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating* **(LRFR)** *of Highway Bridges.*

Three load rating vehicle types are considered: the AASHTO HS20-44, AASHTO Legal Loads, and LRFR HL93. The structural capacity of the controlling bridge component is calculated with the load factor rating (LFR) methods and then compared to the recently introduced reliability-based LRFR rating method. The LRFR method is different from the LFR methods in that it predicts the bridge load rating capacity that results in a probability of bridge failure of 0.0062 (Operating Level). Structural redundancy, the existing condition of the bridge, uncertainties in material behavior, construction quality, and live load magnitudes are all specifically accounted for in the LRFR rating capacity.

Detailed descriptions of the load rating methods are in Chapter 2, and a definition of the structural system is presented in Chapter 3. Chapter 4 describes the analysis assumptions for the bridge structure. Finite element analyses procedures and results of these calculations are outlined in Chapter 5. Chapter 6 presents the demand loads for bridges. Nominal resistance of sections and load rating calculations are presented in Chapters 7 and 8, respectively. Conclusions and recommendations from this investigation are presented in Chapter 9.

2 Description of Load Rating Methods

AASHTO Manual for Condition Evaluation of Bridges

The *AASHTO Manual for Condition Evaluation of Bridges* (2d ed.) will be referred to as the MCEB in this document. The MCEB provides two methods for performing a load rating for a bridge: allowable stress and load factor. In this report, only the load factor method will be considered. The MCEB inventory rating determines the vehicle weight that can cross a bridge for an indefinite period of time. The MCEB operating rating sets the maximum vehicle weight to which a bridge can be subjected before the life of the bridge is shortened.

AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges

The AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges will be referred to as the LRFR in this document. The LRFR employs a probability-based approach for determining the safe load carrying capacity of bridges, in which the statistical distributions of demand load and structural capacity (derived from traffic surveys and experiments) are used to define a probability of bridge failure. The probability that the bridge *will not* fail is called the structural reliability. The LRFR design load rating evaluates the performance of an existing bridge to the AASHTO LRFD HL93 loading (see Appendix A for the definition of an HL93 loading). The HL93 loading was developed to provide uniform reliability across the common range of bridge spans and structural systems on U.S. highways but is generally not consistent with the axle configuration of common truck traffic. This design load rating is performed at two reliability levels corresponding to an inventory rating (lower probability of failure) and operating rating (higher probability of failure).

If a bridge has an LRFR design (operating) load rating less than one, then a legal load rating is conducted with the AASHTO legal loads (see Appendix A). These legal loads are consistent with U.S. truck axle configurations and weights and typically produce a less conservative and more realistic load rating factor to be used for posting on a bridge.

Design codes

The *AASHTO Standard Specifications for Highway Bridges* (17th ed.) will be referred to as AASHTO in this document. This specification describes the structural analysis methods required to calculate structural capacity for MCEB load ratings.

The *AASHTO LRFD Bridge Design Specifications* (3d ed.) will be referred to as LRFD in this document. This specification describes the structural analysis methods required to calculate structural capacity for the LRFR load ratings.

3 Definition of Structural System

Each Lahontan Dam Spillway Bridge consists of five spans with three cast-in-place, continuous concrete arch beams supporting an integrally cast concrete slab. Exterior spans are 50.012 ft, and interior spans are 51.40 ft between bearing lines (Figure 3-1). Each bridge maintains one traffic lane (Figure 3-2 and Figure 3-3). The bridges were constructed in 1915.

Both bridges were inspected in 2009 by the USAED, Sacramento, and given a National Bridge Inventory (NBI) rating of 4 ("Poor Condition"). In other words, the inspection revealed advanced section loss, deterioration at different locations along the full length of the bridge, and severe spalling in all of the three main arch beams and piers.

For this study, the provided copy of construction drawings was used to generate new cross-sectional sketches of the exterior and interior beams with their respective dimensions. Due to some discrepancy in the original drawings, some modifications were made. For example, the slab thickness was assumed to be 15 in. uniformly along the three arch beams; this modification is intended to be conservative. The larger the section, the heavier it is. Also, the slab longitudinal reinforcement pattern, No. 5 bars at a 12-in. spacing, was assumed to be continuous over the beam webs.

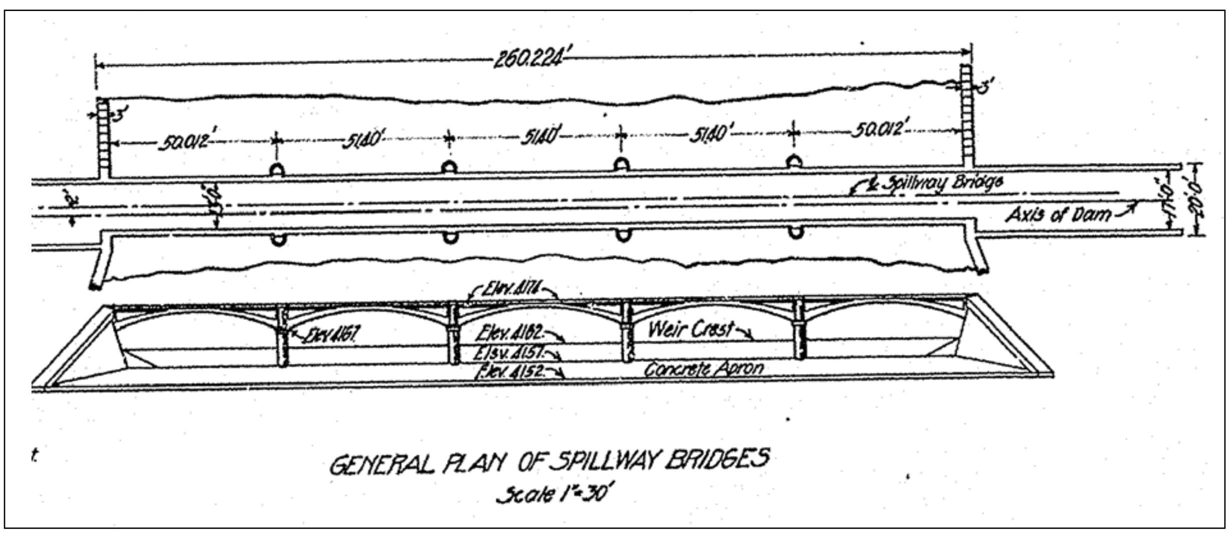


Figure 3-1. Plan and elevation.

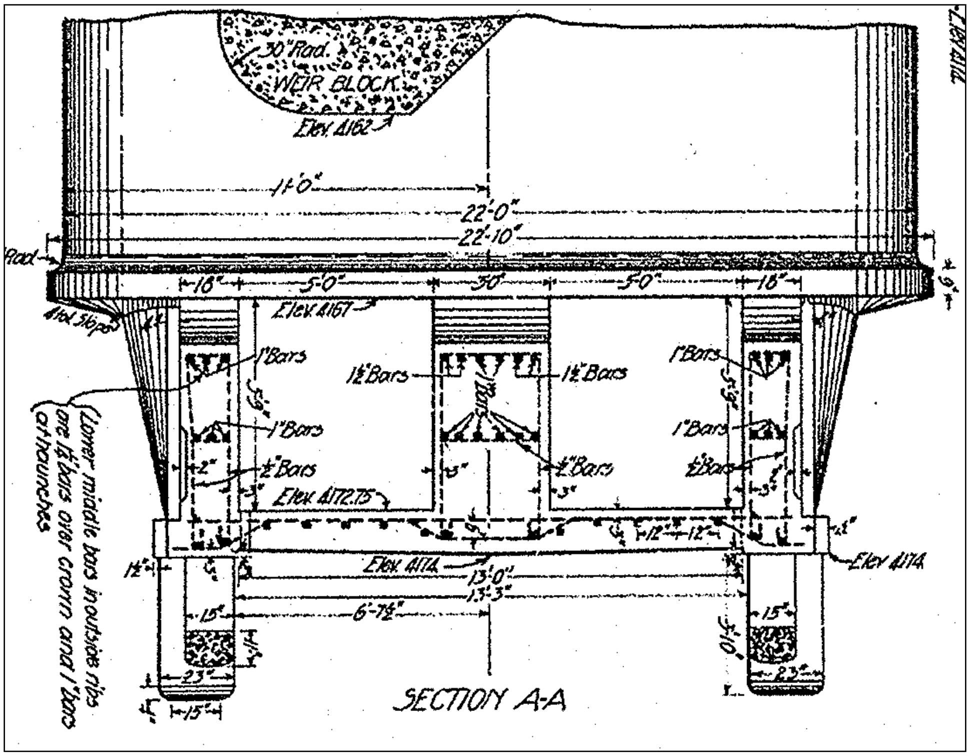


Figure 3-2. Typical bridge cross section.

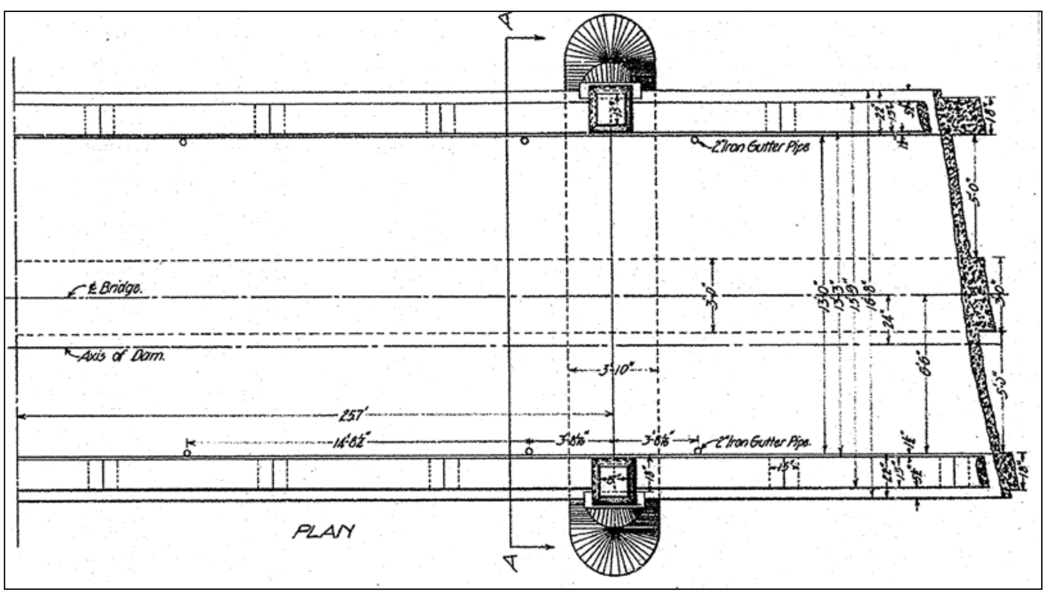


Figure 3-3. Typical bridge plan view.

Figure 3-4 and Figure 3-5 show the location of the steel rebars at the intersection of the arch beams and a pier, herein denoted as the edge of each section. Shear reinforcing is No. 4 stirrups at 2-ft, 5-in. centers, with the first stirrup located 2 ft, 8-7/8 in. from the center of each pier.

The weight effect of the guardrail system was distributed among the three beams on the bridge. Figure 3-5 shows the concrete guardrail dimension. Each span has 4 main concrete posts and 12 intermediate posts.

To be more conservative, the analysis considered the longest span as shown in Figure 3-6.

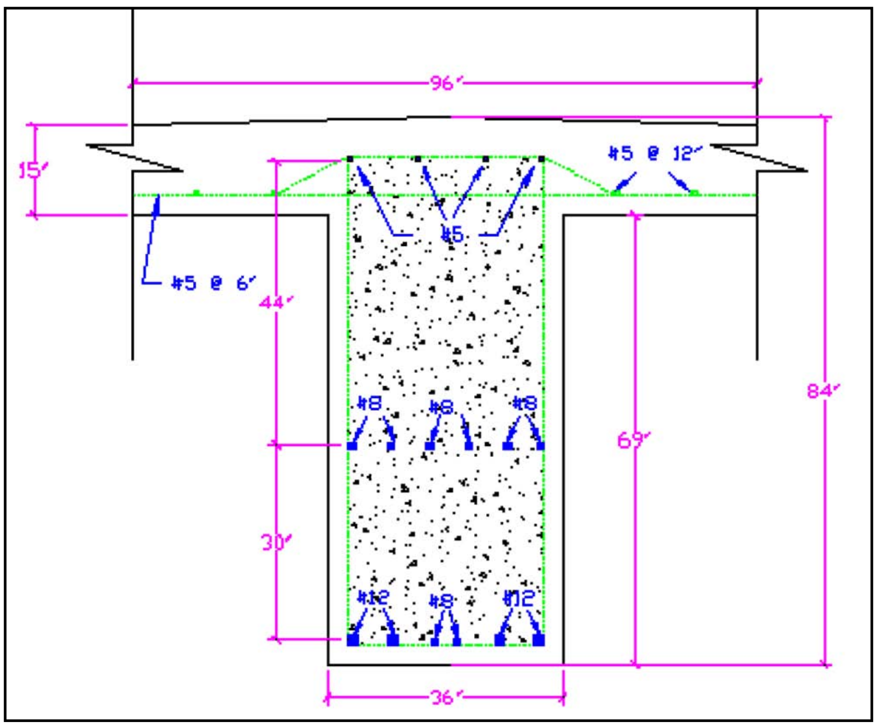


Figure 3-4. Interior beam cross section at the edge.

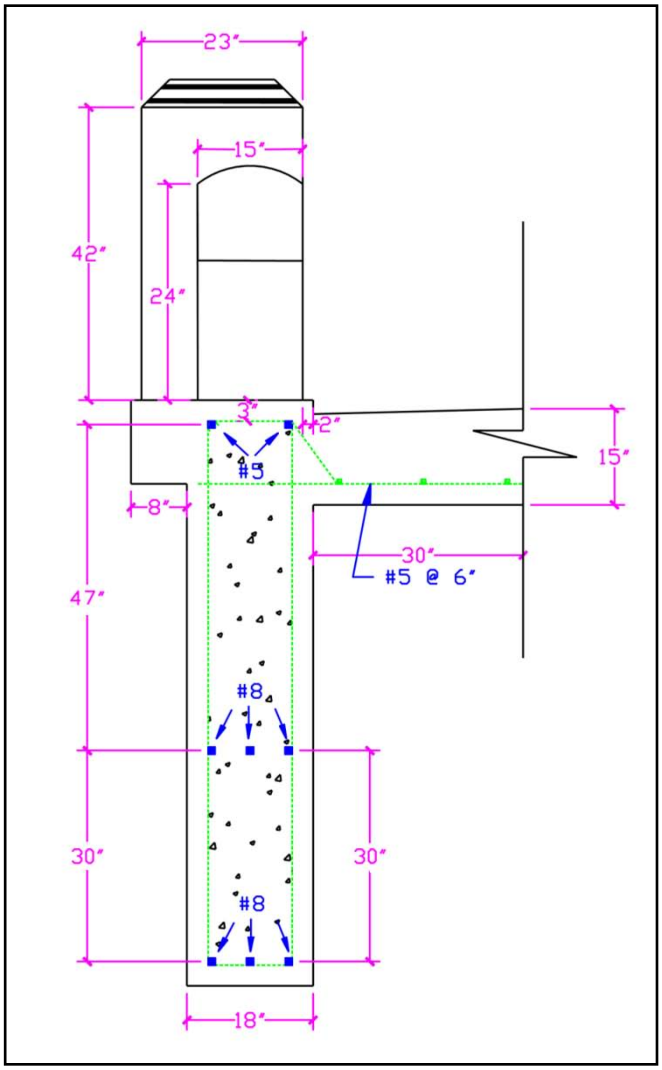


Figure 3-5. Exterior beam cross section at the edge.

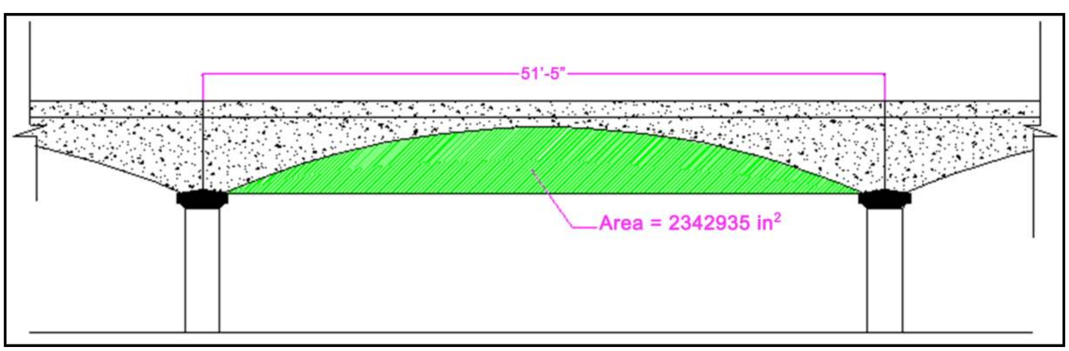


Figure 3-6. Interior span length.

4 Analysis Assumptions

The following assumptions are made in the structural analysis and load rating of this bridge:

1. All bridge dimensions and details are taken from the construction drawing provided in the report of the inspection performed in January 2009 by USAED, Sacramento.
2. Concrete compressive strength is 2500 psi, since the bridge was constructed prior to 1954 (MCEB 6.6.2.4).
3. Concrete unit weight is 150 pcf.
4. Tension yield stress on steel is unknown. For this reason a value of 33,000 psi will be used to perform the analysis, since the bridge was constructed prior to 1954 (MCEB 6.6.2.4).
5. Stress distribution along the beam can be adequately determined by use of elastic finite element analyses of the bridge subjected to dead loads and to moving live loads. Stress distribution at any given section can then be used to determine the dead and the live loads to be used in the load rating of that section.

5 Finite Element Analyses

Since the cross-sectional geometry of an arch beam varies along the span length, a finite element model (FEM) was developed using the structural analysis software SAP2000. This model was used to determine maximum bending moments and shear forces due to loadings on an interior span. For the FEM, beams were considered to be continuous over the piers, and all five spans were modeled. Since beam reinforcing extends into the piers, piers were also modeled in the FEM, and pinned supports were provided at the base of each pier. Gross concrete beam sections and elastic properties were used in all analyses.

The original goal of the analyses was to model the entire three-dimensional (3-D) structure to accurately determine both longitudinal and lateral load distributions. However, due to the size of the final model, the entire structure could not be modeled on a personal computer (PC). Therefore, separate 3-D models were developed for interior and exterior beams, and load distribution among the three beams was calculated using the lever rule. The use of the lever rule will be discussed below under load distribution factors.

Details of the FEM

The sections used to develop the FEM were provided in the design drawings and are described below and illustrated in Figures 5-1 through 5-4.

Span dimensions

L = 51.40 ft	Span length
W = 17.33 ft	Bridge width

Exterior arch ring beam section properties at the span edges

S = 56 in. = 4.66 ft	Width of composite section
A_{Edges} = 2082 in.^{2} = 14.45 ft^2	Cross-sectional area
$I_{B\ Edges}$ = 1392445.3 in.^{4} = 67.15 ft^4	Moment of inertia
$S_{X\ Edges}$ = 27066.54 in.^{3} = 15.66 ft^3	Section modulus
H = 15 in. = 1.25 ft	Slab depth

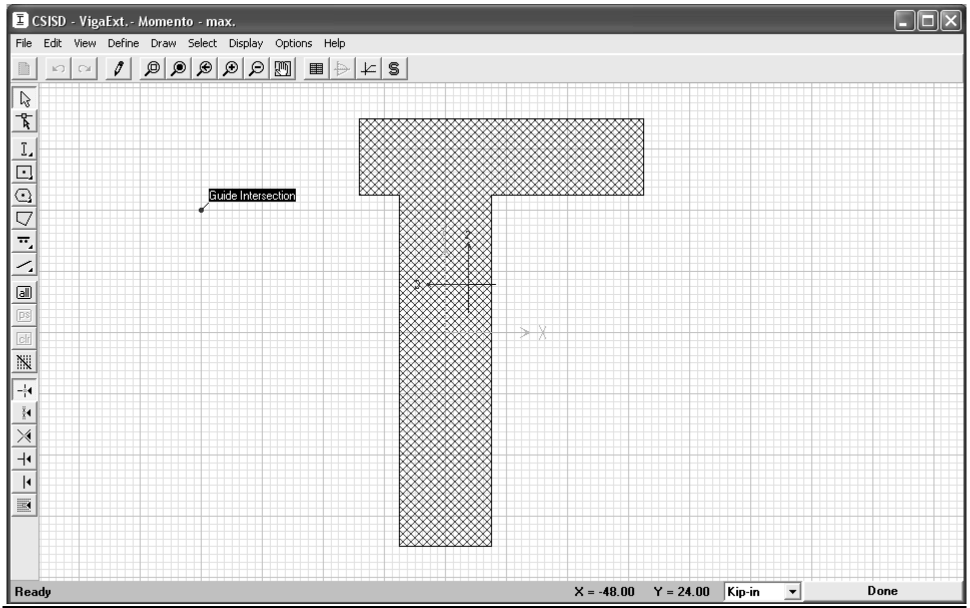


Figure 5-1. Exterior arch ring geometry at span edge.

Exterior arch ring beam section properties at the mid-span

S = 56 in. = 4.66 ft	Width of composite section
$A_{Mid\text{-}span}$ = 1006.86 in.^{2} = 6.99 ft^2	Cross-sectional area
$I_{B\ Mid\text{-}span}$ = 37444.34 in.^{4}= 1.80 ft^4	Moment of inertia
$S_{X\ Mid\text{-}span}$ = 2537.05 in.^{3}= 1.46 ft^3	Section modulus
H = 15 in. = 1.25 ft	Slab depth

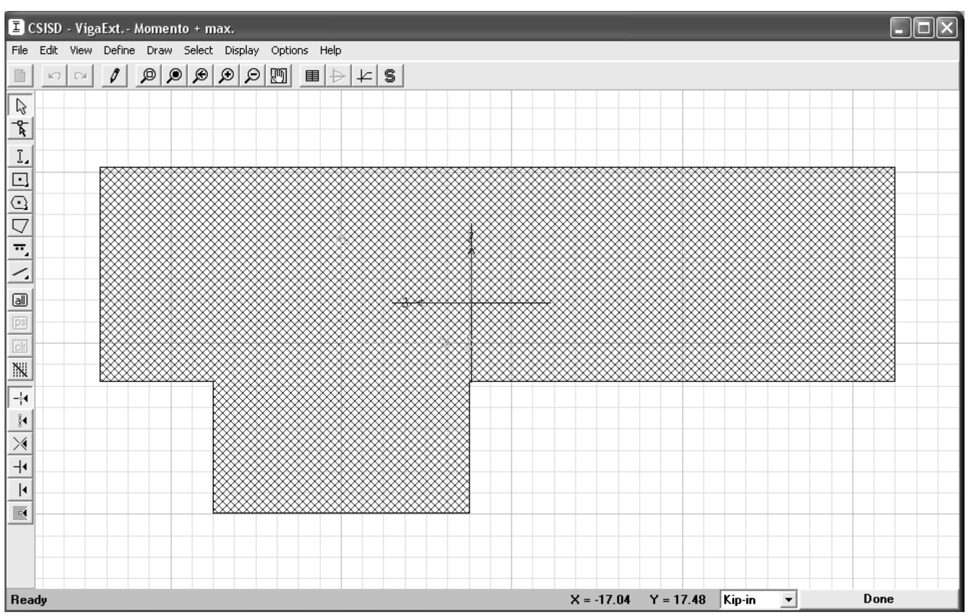


Figure 5-2. Exterior arch ring geometry at mid-span.

Interior arch ring beam section properties at the span edges

S=8 ft	Width of composite section
A_{Edges} =3924 in^2 = 27.25 ft^2	Cross-sectional area
$I_{B\ Edges}$ =2620518.2 in^4=126.38 ft^4	Moment of inertia
$S_{X\ Edges}$ =52501.88 in^3=30.38 ft^3	Section modulus
H=15 in=1.25 ft	Slab depth

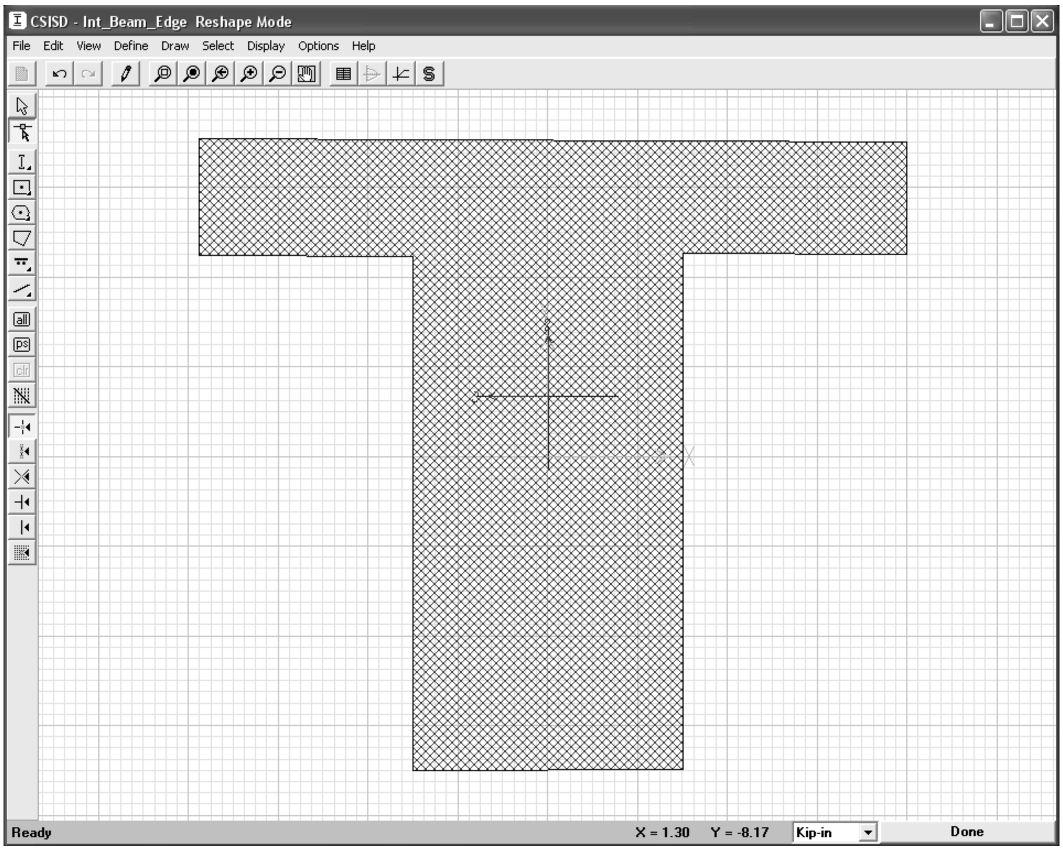


Figure 5-3. Interior arch ring geometry at span edge.

Interior arch ring beam section properties at the mid-span

S=8 ft	Width of composite section
$A_{B\ Mid\text{-}span}$ =1704 in^2 = 11.83 ft^2	Cross-sectional area
$I_{B\ Mid\text{-}span}$ =65839.89 in^4=38.1 ft^4	Moment of inertia
$S_{X\ B\ Mid\text{-}span}$ =4551.74 in^3=2.63 ft^3	Section modulus
H=15 in. =1.25 ft	Slab depth

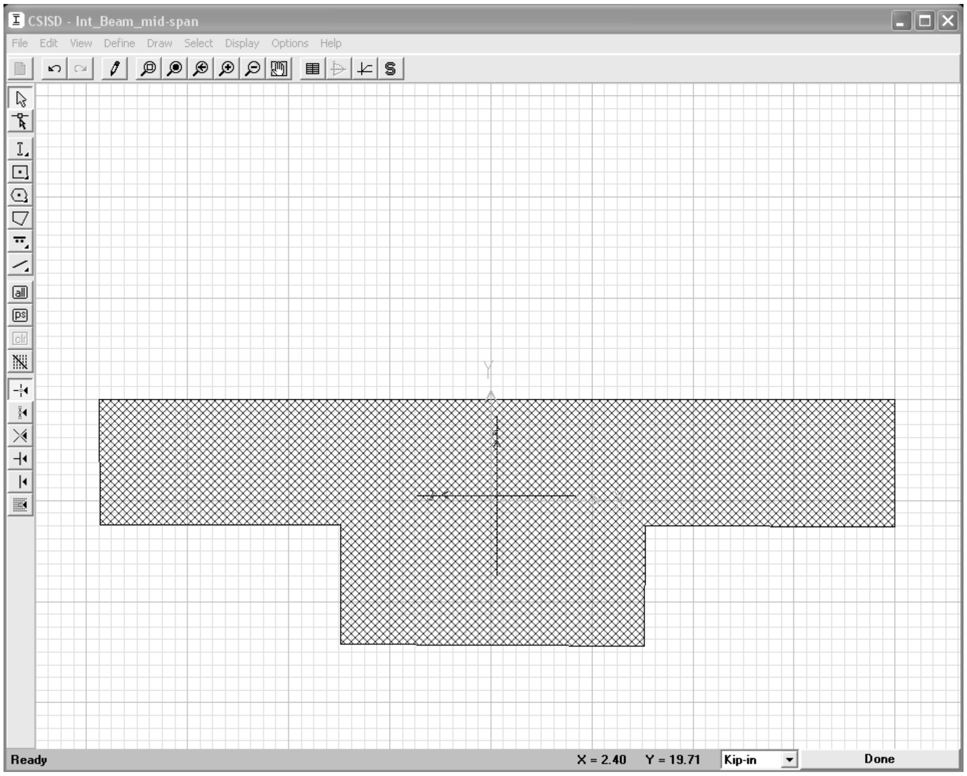


Figure 5-4. Interior arch ring geometry at mid-span.

The following additional information was provided in the structural drawings and was used to develop the FEMs:

Diameter at the base of the arch	61.05 ft
Clear span for an interior section	48.4 ft
Pier width	3 ft
Pier height	10 ft

Both exterior and interior T-beam sections meet the flange versus web criteria ratios provided in the American Concrete Institute (ACI) 318 code on section 8.10.

Conservatively, the thickness of the pier section used in the analyses was the same as that of the beam web above. The 3-D shell models of each beam are shown in Figure 5-5 and 5-6.

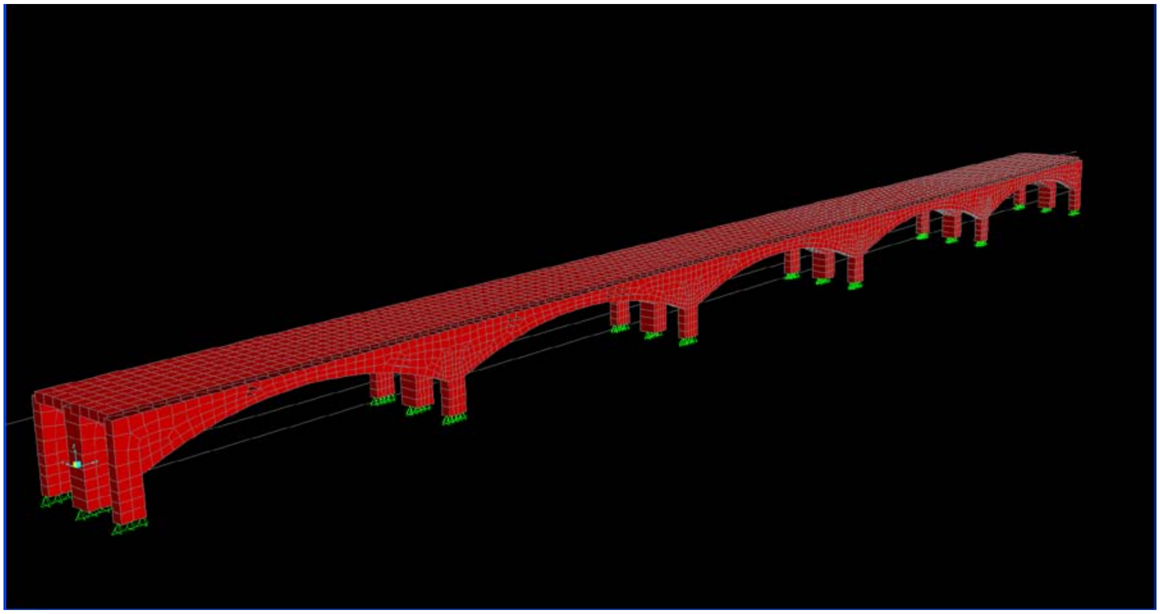


Figure 5-5. Finite element beam models.

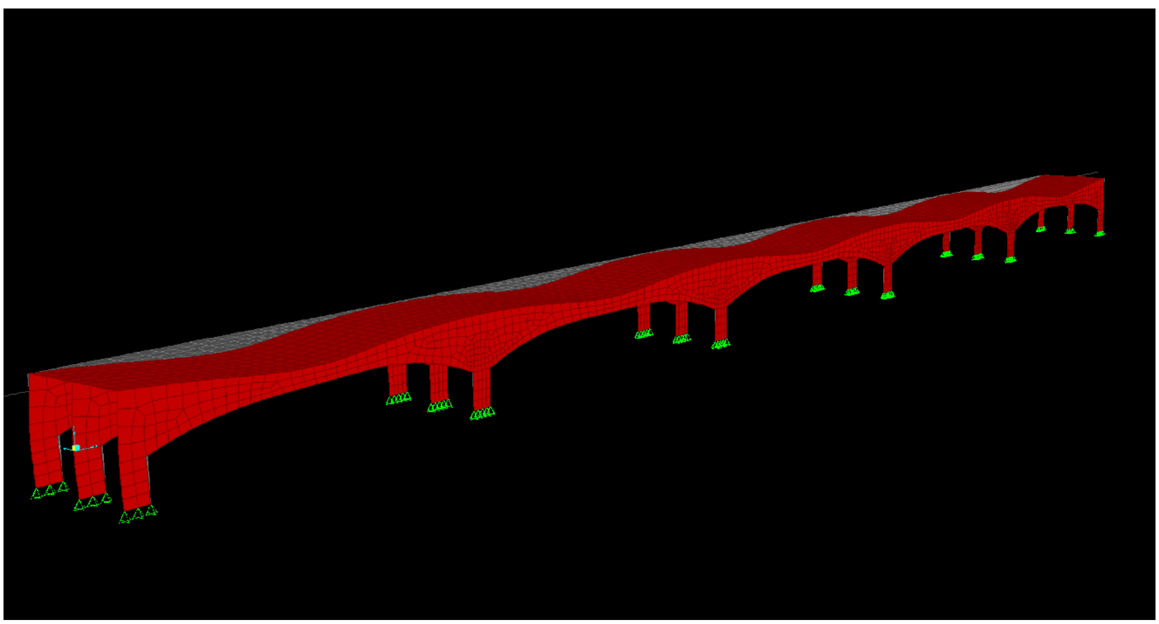


Figure 5-6. Deformed shape due to dead load.

Dead load (DL) + superimposed dead load (SDL) analysis

Self-weight (dead load)

Self-weight for the DL analysis is provided by using a unit weight for the material of 150 pcf.

Bridge railings (SDL)

The SDL is provided by the bridge railings. Weight of the bridge railings is assumed to be distributed evenly over the three-arch ring, cast-in-place units. The calculation for w_{SDL} is provided in Appendix B.

$$W_{SDL} = 0.25 \frac{kips}{ft}$$

Analysis results

A typical stress distribution from an interior span DL + SDL analysis is shown in Figure 5-7. The decision to calculate moments, shears and capacities at four approximately equally spaced sections along one half of an interior span was based on the typical stress distribution, and provides two sections for study in the negative-moment region and two in the positive-moment region. Sections used in the calculations were at approximately one third of the distance between the support edge and mid-span. If the center of a support is considered to be at 0.0 ft, section locations are at 1.5 ft, 9 ft, 16.67 ft and 25.7 ft. Section locations are shown in Figure 5-8. The SAP2000 calculations of negative moment at the edge section and positive moment at the mid-span section from the DL + SDL analysis of the exterior beam are shown in Figure 5-9 and Figure 5-10, respectively.

Results from the DL + SDL analysis were used to calculate the bending moments and shear forces in the exterior and interior beams at the four sections described above. The effect of the arch shape in the beams is to decrease the positive bending moment at the mid-span and increase the negative moment near the supports.

Table 5-1 and Table 5-2 show a summary of the results for the exterior and interior beams, respectively, due the self-weight and superimposed dead load.

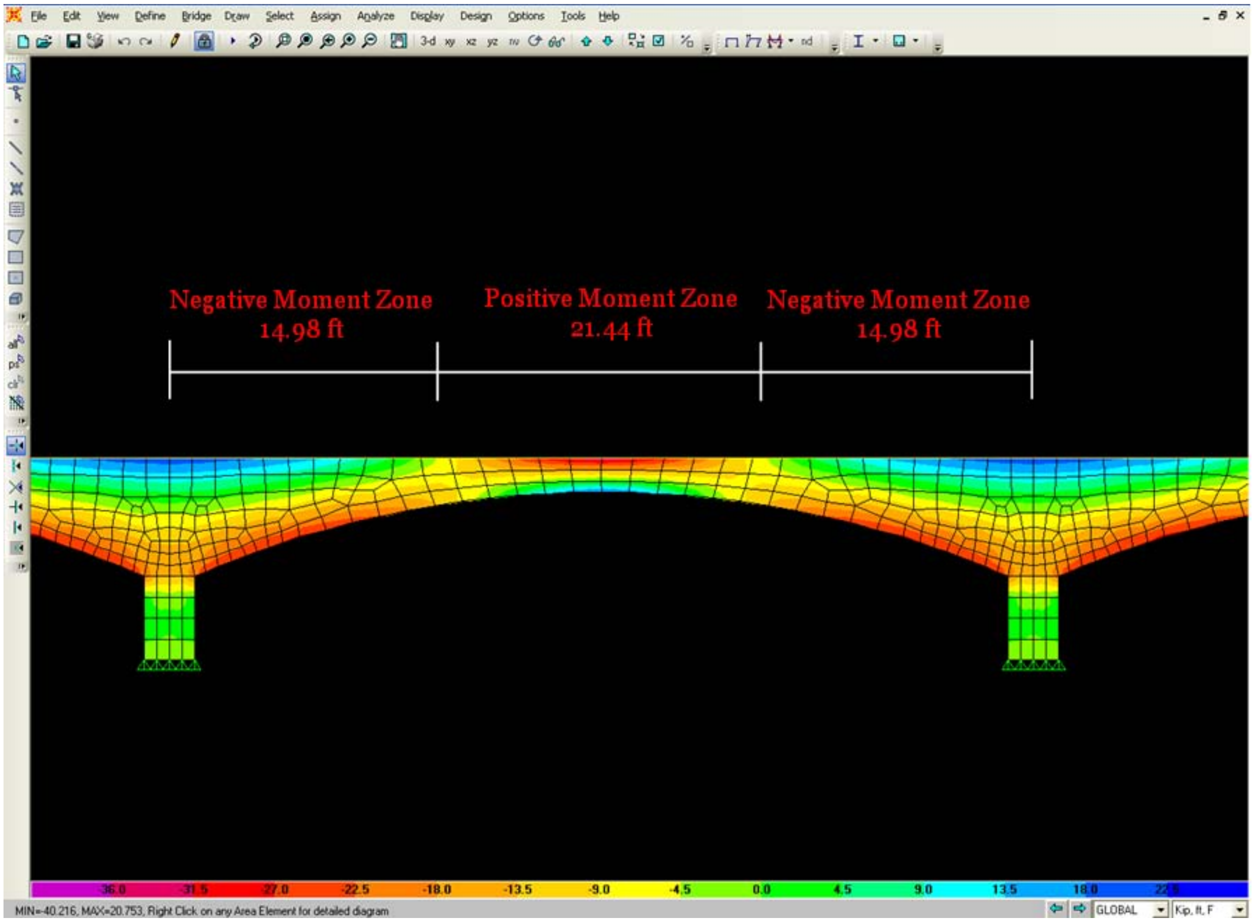


Figure 5-7. Stress distribution due to DL and SDL.

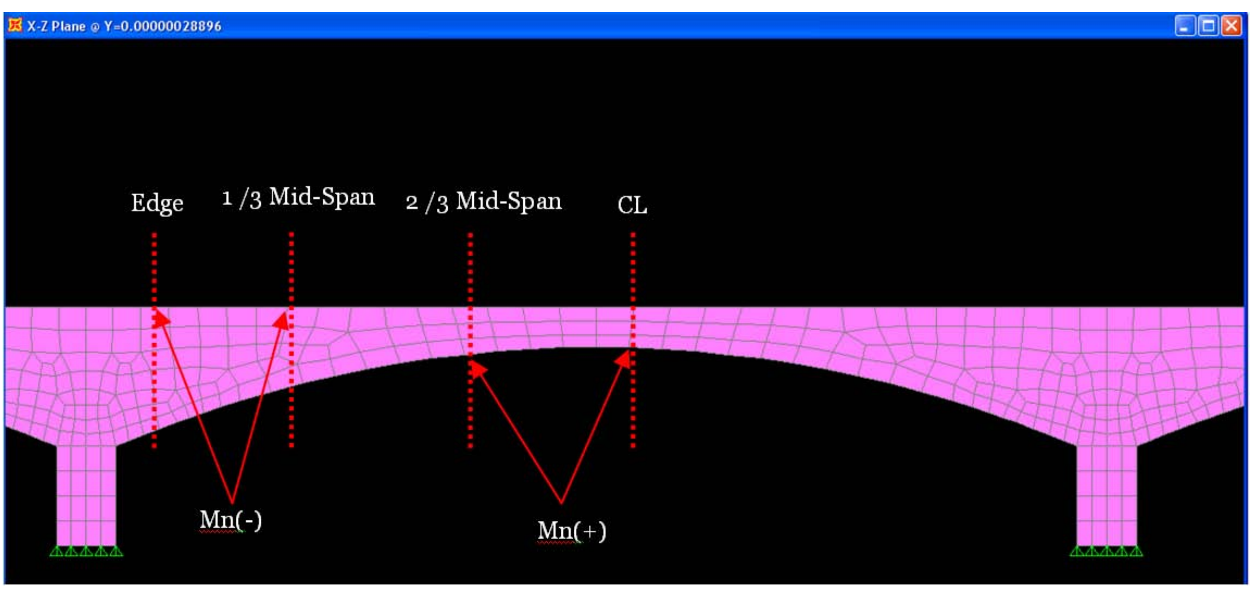


Figure 5-8. Locations for calculation of moments, shear and capacity.

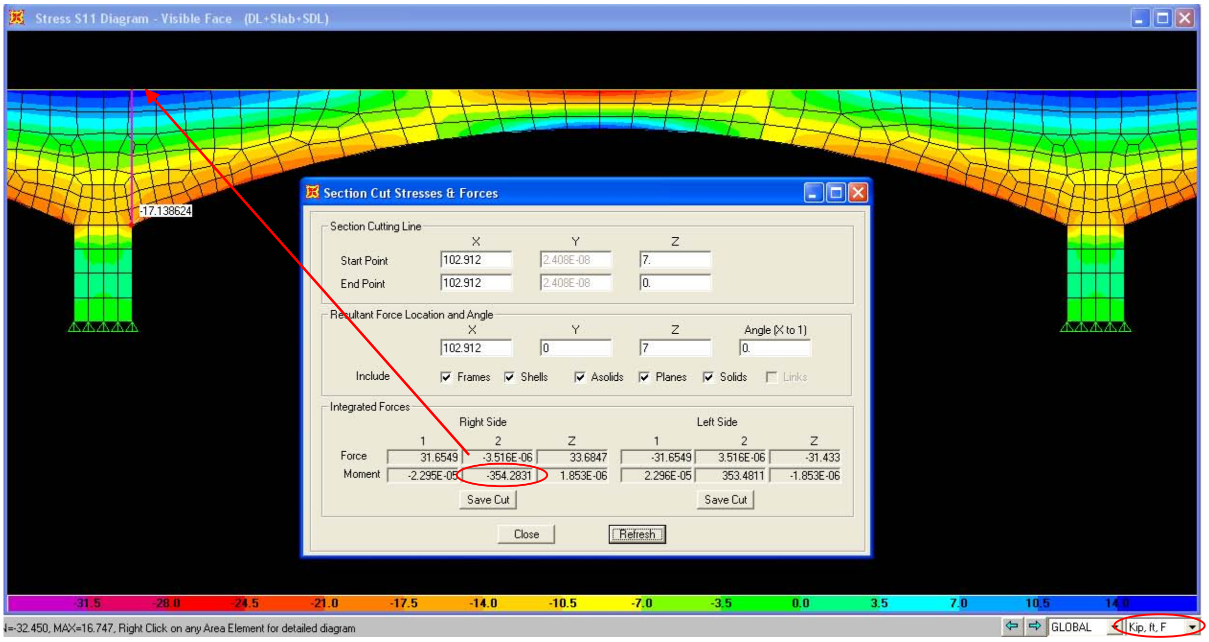


Figure 5-9. Maximum negative moment at the edge section of the exterior beam from the DL + SDL analysis.

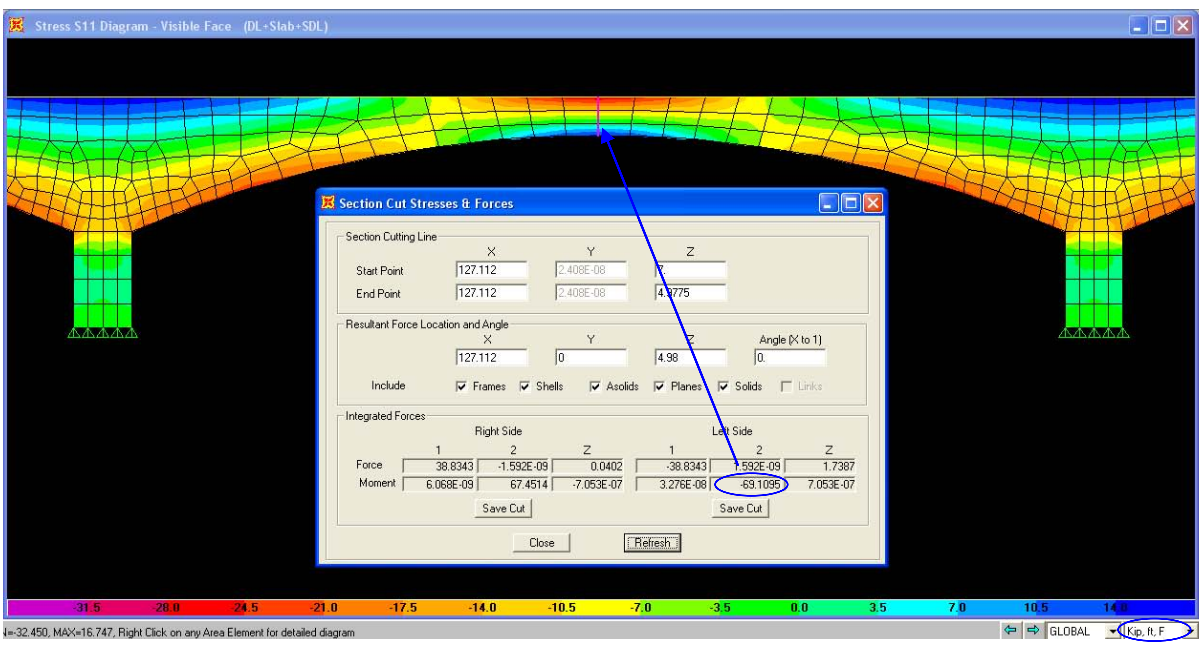


Figure 5-10. Maximum positive moment at the mid-span section, exterior beam from the DL + SDL analysis.

Table 5-1. Summary of bending moment and shear force of the exterior beam due to DL and SDL.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-354.28	-164.88	40.11	69.10
Shear Force (kip)	33.68	21.42	11.89	1.73

Table 5-2. Summary of bending moment and shear force of the interior beam due to DL and SDL.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-618.49	288.44	66.81	114.34
Shear Force (kip)	59.26	37.15	20.22	3.31

Live load

General

The live load moments and shears below are the maximum values calculated from the FEM analyses results at the four sections considered. In each analysis, the loading was simulated by moving the axle loads of the rating vehicle along the center of a beam web of a 51.4-ft interior span. These values must be divided by 2 to produce the wheel-line loadings used in the load factor rating (LFR).

Refer to Appendix A for a description of each loading case.

SAP2000 live load (LL), moment and shear

HS20-44

The following figures show the SAP2000 calculation of moment and shear for the maximum negative moment, maximum positive moment, and maximum shear in an exterior beam.

At the edge section (Figure 5-11):

$$M_{LL} = -591.77\ ft\text{-}k$$
$$V_{LL} = 59.55\ k$$

At the mid-span section (Figure 5-12):

$$M_{LL} = 209.01\ ft\text{-}k$$

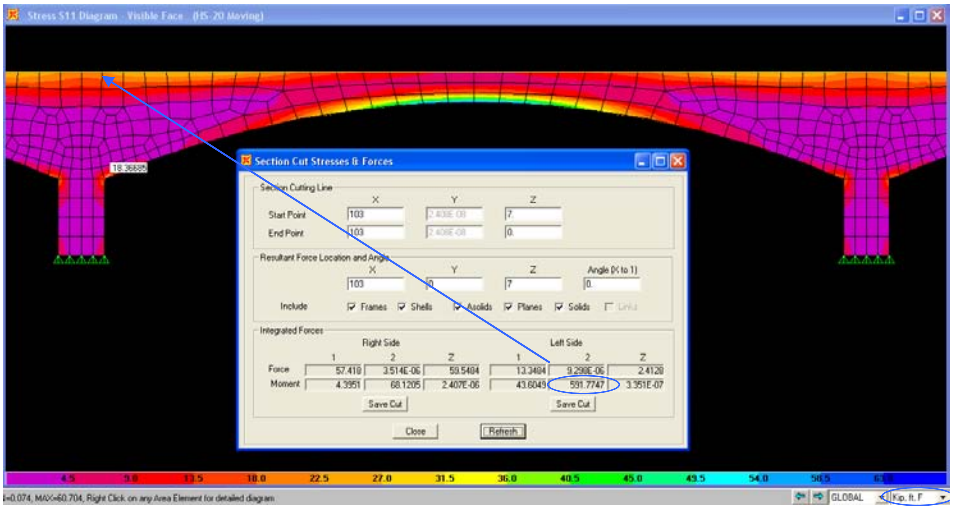


Figure 5-11. Maximum bending moment at edge section of exterior beam for HS20-44 axle loading.

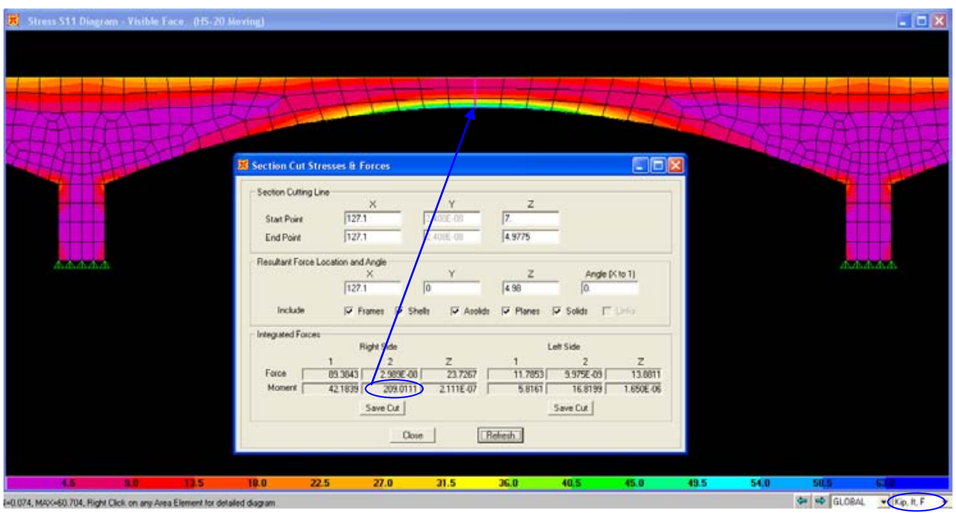


Figure 5-12. Maximum mid-span moment for the exterior arch beam from HS20 analysis

Table 5-3 contains maximum bending moments and shears at each of the four sections from the HS20-44 loading applied to exterior and interior beams.

Table 5-3. Maximum moments and shear forces due to HS20-44 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-591.77	-324.96	143.32	209.01
Shear Force (kip)	59.55	48.97	38.09	23.72

Type 3

Table 5-4 contains maximum bending moments and shears at each of the four sections from the Type 3 loading applied to exterior and interior beams.

Table 5-4. Summary of bending moments and shear forces due to Type 3 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-439.28	-251.77	97.11	159.37
Shear Force (kip)	43.29	36.44	29.36	17.20

Type L3S2

Table 5-5 contains maximum bending moments and shears at each of the four sections from the Type 3S2 loading applied to exterior and interior beams.

Table 5-5. Summary of bending moments and shear forces due to Type 3S2 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-420.24	-237.44	85.98	147.77
Shear Force (kip)	40.73	34.51	28.13	15.98

Type L3-3

Table 5-6 contains maximum bending moments and shears at each of the four sections from the Type L3-3 loading applied to exterior and interior beams.

Table 5-6. Summary of bending moments and shear forces due to Type L3-3 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-379.33	-209.76	84.39	131.49
Shear Force (kip)	39.62	31.55	24.44	14.43

Design tandem

Table 5-7 contains maximum bending moments and shears at each of the four sections from the tandem loading applied to exterior and interior beams.

Table 5-7. Summary of bending moments and shear forces due to tandem loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-541.14	-318.27	142.81	220.04
Shear Force (kip)	46.09	44.0	38.33	24.07

Lane load

Table 5-8 contains maximum bending moments and shears at each of the four sections due to the uniform lane loading applied to exterior and interior beams of 0.64 kip/ft.

Table 5-8. Summary of bending moments and shear forces due the lane load of 0.64 kip/ft.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-185.05	-91.35	20.55	38.69
Shear Force (kip)	15.58	10.95	6.8	1.12

HL93

Table 5-9 contains maximum bending moments and shears at each of the four sections from the HL93 design truck plus lane loading applied to exterior and interior beams.

Table 5-9. Summary of bending moments and shear forces design HL93 loading.

	Location along the beam			
Load	**Edge**	**1/3 Mid-span**	**2/3 Mid-span**	**Mid-span**
Bending Moment (kip-ft)	-480.94	-253.83	92.21	148.71
Shear Force (kip)	45.35	35.43	25.84	13.15

6 Demand Loads

Demand loads are provided by multiplying each load by a distribution factor (DF) and impact factor (IM). The (MCEB and LRFR distribution and impact factors are based on statistical analyses of notional vehicles used to represent typical highway loads. To calculate demand loads for a load factor rating (LFR), the governing codes are those in AASHTO and the MCEB. To calculate demand loads for a load and resistance factor rating, the governing codes are the LRFR and the LRFD.

Distribution factor

AASHTO and the LRFD provide simplified DF for moment in cast-in-place concrete T-beams. Since the bridge has only three concrete T-beams, the equation from the LRFD Tables 4.6.2.2.2 and 4.6.2.2.3 does not apply. Therefore, the lever rule was used to calculate the DF for both the exterior and interior beams for the LRF rating. The lever rule should provide conservative results for interior beams, and the interior beam does not control this calculation. Also, this is only a demonstration of the LRFR method. Load ratings are based on the LF method.

For the LF rating:

Distribution factor for an interior T-beam subject to one lane of traffic is

DF = S/6.5; S is spacing of beams in feet (AASHTO Table 3.23.1)

DF = S/6.5 = 7.25/6.5 = 1.15

Distribution factor for an exterior T-beam (Figure 6-1) is given by the lever rule and, therefore, is the same as that for the LRF rating.

For the LRF rating:

Exterior beam – one lane loaded:

Use the lever rule with first wheel load positioned at 1 ft from the parapet.

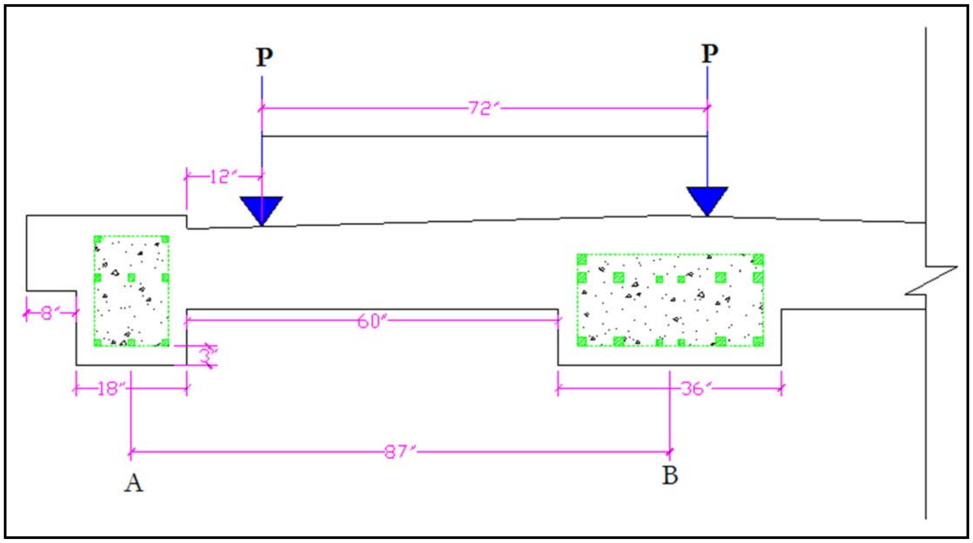


Figure 6-1. Distribution factor for the exterior beam.

$$\sum M_B = 0$$

$$6P - 66P + 87R_A = 0$$

$$DF_{Ext} = \frac{66 + 87}{87} = 0.69$$

Interior beam – one lane loaded:

Use the lever rule with one of the wheel loads positioned in the centerline of the interior beam (Figure 6-2).

$$\sum M_A = 0$$

$$15P + 87P - 87R_B = 0$$

$$DF_{Ext} = \frac{15 + 87}{87} = 1.17$$

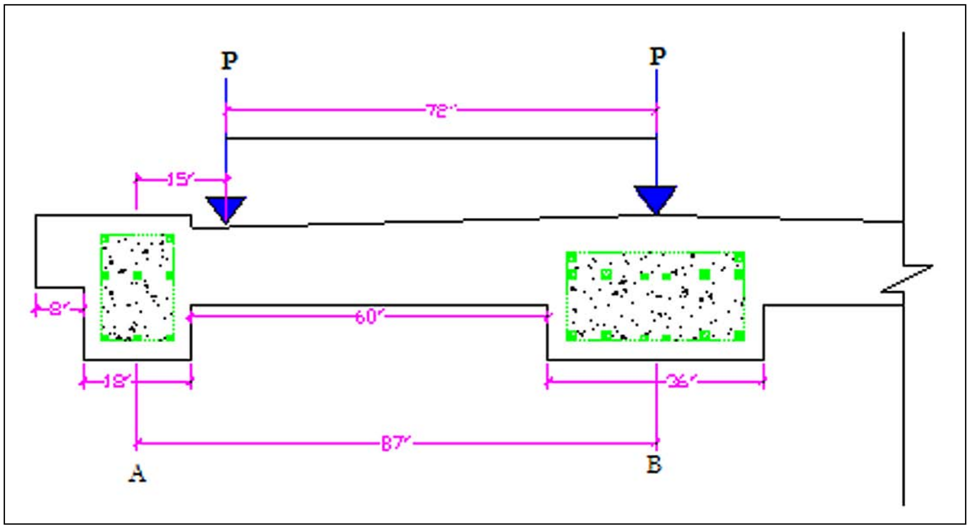


Figure 6-2. Distribution factor for the interior beam.

Note that the DF for shear is given by the lever rule. For this reason, only one calculation was performed.

Dynamic impact factors

The MCEB dynamic impact factor for live load is

$$I = 1 + \frac{50}{L + 125} \leq 1.30 \qquad \text{(MCEB 6.7.4)}$$

$$I = 1 + \frac{50}{51.4 + 125} = 1.28 < 1.30$$

$$\therefore I = 1.28$$

The LRFR dynamic impact factor for live load is

$$IM = 1.33 \qquad \text{(LRFR 6.4.4.3)}$$

Compute maximum live load effects

Since the MCEB and LRFR provide different values for DF and IM, separate calculations must be performed to calculate demand loads for the two methods. The LFR calculations are based on wheel line loads. Normally, the LRFD distribution factors provide a distribution for each girder based on axle loads. Since the lever rule was used to calculate distribution factors for the LRFR calculation, LRFR demand loads were based on ½ of axle loads, just as in the LFR calculations.

Live load moments and shear are those produced from the wheel line of the rating vehicle on a 51.4-ft span. The maximum live load shear (without impact) caused by one wheel line of trucks is ½ the value obtained from the computer model. This value is multiplied by a dynamic impact factor, *I*, and the lever rule distribution factor to determine the maximum live load shear:

Exterior beam, LFR

Using the shear force at the edge section from Table 5-3 through Table 5-9,

$$V_{LL} = V_{HS20} \times I \times DF = \frac{59.55}{2} kips \times 1.28 \times 0.69 = 26.29\, kips$$
$$V_{LL} = V_{Type3} \times I \times DF = \frac{43.29}{2} kips \times 1.28 \times 0.69 = 19.12\, kips$$
$$V_{LL} = V_{Type3S2} \times I \times DF = \frac{40.73}{2} kips \times 1.28 \times 0.69 = 17.99\, kips$$
$$V_{LL} = V_{Type3-3} \times I \times DF = \frac{39.62}{2} kips \times 1.28 \times 0.69 = 17.49\, kips$$
$$V_{LL} = V_{Tandem} \times I \times DF = \frac{46.09}{2} kips \times 1.28 \times 0.69 = 20.35\, kips$$

Interior beam, LFR

Using the shear force at the edge section from Table 5-3 through Table 5-9,

$$V_{LL} = V_{HS20} \times I \times DF = \frac{59.55}{2} kips \times 1.28 \times 1.17 = 44.59\, kips$$
$$V_{LL} = V_{Type3} \times I \times DF = \frac{43.29}{2} kips \times 1.28 \times 1.17 = 32.42\, kips$$
$$V_{LL} = V_{Type3S2} \times I \times DF = \frac{40.73}{2} kips \times 1.28 \times 1.17 = 30.50\, kips$$

$$V_{LL} = V_{Type3-3} \times I \times DF = \frac{39.62}{2} kips \times 1.28 \times 1.17 = 29.67\ kips$$

$$V_{LL} = V_{Tandem} \times I \times DF = \frac{46.09}{2} kips \times 1.28 \times 1.17 = 34.51\ kips$$

Exterior beam, LRFR

Using the shear force at the edge section from Table 5-3 through Table 5-9,

$$V_{LL} = V_{HS20} \times I \times DF = \frac{59.55}{2} kips \times 1.33 \times 0.69 = 27.32\ kips$$

$$V_{LL} = V_{Type3} \times I \times DF = \frac{43.29}{2} kips \times 1.33 \times 0.69 = 19.87\ kips$$

$$V_{LL} = V_{Type3S2} \times I \times DF = \frac{40.73}{2} kips \times 1.33 \times 0.69 = 18.69\ kips$$

$$V_{LL} = V_{Type3-3} \times I \times DF = \frac{39.62}{2} kips \times 1.33 \times 0.69 = 18.17\ kips$$

$$V_{LL} = V_{Tandem} \times I \times DF = \frac{46.09}{2} kips \times 1.33 \times 0.69 = 21.15\ kips$$

$$V_{LL} = V_{HL-93} \times I \times DF = \left(\frac{59.55}{2} kips \times 1.33 + 15.58 kips\right) \times 0.69 = 38.07\ kips$$

Interior beam, LRFR

Using the shear force at the edge section from Table 5-3 through Table 5-9,

$$V_{LL} = V_{HS20} \times I \times DF = \frac{59.55}{2} kips \times 1.33 \times 1.17 = 46.33\ kips$$

$$V_{LL} = V_{Type3} \times I \times DF = \frac{43.29}{2} kips \times 1.33 \times 1.17 = 33.69\ kips$$

$$V_{LL} = V_{Type3S2} \times I \times DF = \frac{40.73}{2} kips \times 1.33 \times 1.17 = 31.69\ kips$$

$$V_{LL} = V_{Type3-3} \times I \times DF = \frac{39.62}{2} kips \times 1.33 \times 1.17 = 30.81\ kips$$

$$V_{LL} = V_{Tandem} \times I \times DF = \frac{46.09}{2} kips \times 1.33 \times 1.17 = 35.86\ kips$$

$$V_{LL} = V_{HL-93} \times I \times DF = \left(\frac{59.55}{2} kips \times 1.33 + 15.58 kips\right) \times 1.17 = 64.56\ kips$$

The computer model provides the maximum live load moment caused by the truck axles. This value is divided by 2 to get wheel line loads, then multiplied by the multiple wheel line factor *DF* and the IM factor to determine the maximum live load moment:

Exterior beam, LFR

Using the bending moments at the edge section from Table 5-3 through Table 5-9,

$$M_{LL} = (M+I)_{HS20} \times DF = \frac{-591.77}{2} kip \cdot ft \times 1.28 \times 0.69 = -261.33\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3} \times DF = \frac{-439.28}{2} kip \cdot ft \times 1.28 \times 0.69 = -193.99\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3S2} \times DF = \frac{-420.24}{2} kip \cdot ft \times 1.28 \times 0.69 = -185.58\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3-3} \times DF = \frac{-379.33}{2} kip \cdot ft \times 1.28 \times 0.69 = -167.51\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Tandem} \times DF = \frac{-541.14}{2} kip \cdot ft \times 1.28 \times 0.69 = -238.97\, kip \cdot ft$$

Interior beam, LFR

Using the bending moments at the edge section from Table 5-3 through Table 5-9,

$$M_{LL} = (M+I)_{HS20} \times DF = \frac{-591.77}{2} kip \cdot ft \times 1.28 \times 1.17 = -443.12\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3} \times DF = \frac{-439.28}{2} kip \cdot ft \times 1.28 \times 1.17 = -328.94\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3S2} \times DF = \frac{-420.24}{2} kip \cdot ft \times 1.28 \times 1.17 = -314.68\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3-3} \times DF = \frac{-379.33}{2} kip \cdot ft \times 1.28 \times 1.17 = -284.04\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Tandem} \times DF = \frac{-541.14}{2} kip \cdot ft \times 1.28 \times 1.17 = -405.21\, kip \cdot ft$$

Exterior beam, LRFR

Using the bending moments at the edge section from Table 5-3 through Table 5-9,

$$M_{LL} = (M+I)_{HS20} \times DF = \frac{-591.77}{2} kip \cdot ft \times 1.33 \times 0.69 = -271.54\, kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3} \times DF = \frac{-439.28}{2} kip \cdot ft \times 1.33 \times 0.69 = -201.57 kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3S2} \times DF = \frac{-420.24}{2} kip \cdot ft \times 1.33 \times 0.69 = -192.83\ kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3-3} \times DF = \frac{-379.33}{2} kip \cdot ft \times 1.33 \times 0.69 = -174.05\ kip \cdot ft$$

$$M_{LL} = (M+I)_{Tandem} \times DF = \frac{-541.14}{2} kip \cdot ft \times 1.33 \times 0.69 = -248.30\ kip \cdot ft$$

$$M_{LL} = (M+I)_{HL-93} \times DF = \left(\frac{-591.77}{2} kip \cdot ft \times 1.33 + -185.05 kip.ft \right) \times 0.69 = -399.22\ kip \cdot ft$$

Interior beam, LRFR

Using the bending moments at the edge section from Table 5-3 through Table 5-9,

$$M_{LL} = (M+I)_{HS20} \times DF = \frac{-591.77}{2} kip \cdot ft \times 1.33 \times 1.17 = -460.44\ kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3} \times DF = \frac{-439.28}{2} kip \cdot ft \times 1.33 \times 1.17 = -341.79 kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3S2} \times DF = \frac{-420.24}{2} kip \cdot ft \times 1.33 \times 1.17 = -326.97\ kip \cdot ft$$

$$M_{LL} = (M+I)_{Type3-3} \times DF = \frac{-379.33}{2} kip \cdot ft \times 1.33 \times 1.17 = -295.13\ kip \cdot ft$$

$$M_{LL} = (M+I)_{Tandem} \times DF = \frac{-541.14}{2} kip \cdot ft \times 1.33 \times 1.17 = -421.03\ kip \cdot ft$$

$$M_{LL} = (M+I)_{HL-93} \times DF = \left(\frac{-591.77}{2} kip \cdot ft \times 1.33 + -185.05 kip.ft \right) \times 1.17 = -676.93\ kip \cdot ft$$

HS20-44

Complete results for LFR demand load at the four sections of the exterior and interior beams are provided in Table 6-1 and Table 6-2, respectively. Complete results for LRFR demand load are provided in Table 6-3 and Table 6-4.

Table 6-1. Summary of LFR demand bending moments and shear forces in the exterior beam due to HS20-44 loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-261.33	-143.50	63.29	92.30
Shear Force (kip)	26.29	21.62	16.82	10.49

Table 6-2. Summary of LFR demand bending moments and shear forces in the interior beam due to HS20-44 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-443.12	-243.33	107.32	156.51
Shear Force (kip)	44.58	36.68	28.52	17.79

Table 6-3. Summary of LRFR demand bending moments and shear forces in the exterior beam due to HS20-44 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-271.53	149.11	65.76	95.90
Shear Force (kip)	27.32	22.47	17.48	10.90

Table 6-4. Summary of LRFR demand bending moments and shear forces in the interior beam due to HS20-44 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-460.43	-252.84	111.51	162.62
Shear Force (kip)	46.33	38.10	29.64	18.49

AASHTO legal loads

Type 3

The Type 3 legal load produces the maximum shear and moment in the relatively short span of 51.4 ft considered here. (Note that the AASHTO Type 3S2 legal load will control for medium span lengths (60 ft–90 ft) and Type 3-3 legal loads will control for longer spans (<90 ft).

Complete results for LFR demand load at the four sections of the exterior and interior beams are provided in Table 6-5 and Table 6-6, respectively. Complete results for LRFR demand load are provided in Table 6-7 and Table 6-8.

Table 6-5. Summary of LFR demand bending moments and shear forces in the exterior beam due to Type 3 loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-193.99	-111.18	42.88	70.38
Shear Force (kip)	19.12	16.09	12.96	7.59

Table 6-6. Summary of LFR demand bending moments and shear forces in the interior beam due to Type 3 loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-328.94	-188.53	72.72	119.34
Shear Force (kip)	32.42	27.29	21.98	12.88

Table 6-7. Summary of LRFR demand bending moments and shear forces in the exterior beam due to Type 3 loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-201.56	-115.52	44.56	73.13
Shear Force (kip)	19.86	16.72	16.47	7.89

Table 6-8. Summary of LRFR demand bending moments and shear forces in the interior beam due to Type 3 loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-341.78	-195.89	75.56	123.99
Shear Force (kip)	33.68	28.35	22.84	13.38

Type L3S2

Complete results for LFR demand load at the four sections of the exterior and interior beams are provided in Table 6-9 and Table 6-10, respectively.

Complete results for LRFR demand load are provided in Table 6-11 and Table 6-12.

Table 6-9. Summary of LFR demand bending moments and shear forces in the exterior beam due to Type 3S2 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-185.58	-104.85	37.97	65.25
Shear Force (kip)	17.99	15.24	12.42	7.06

Table 6-10. Summary of LFR demand bending moments and shear forces in the interior beam due to Type 3S2 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-314.68	-177.80	64.38	110.65
Shear Force (kip)	30.50	25.84	21.06	11.96

Table 6-11. Summary of LRFR demand bending moments and shear forces in the exterior beam due to Type 3S2 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-192.83	-108.98	39.45	67.80
Shear Force (kip)	18.69	15.83	12.91	7.33

Table 6-12. Summary of LRFR demand bending moments and shear forces in the interior beam due to Type 3S2 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-326.97	-184.74	66.90	114.97
Shear Force (kip)	31.69	26.85	21.87	12.43

Type 3-3

Complete results for LFR demand load at the four sections of the exterior and interior beams are provided in Table 6-13 and Table 6-14, respectively. Complete results for LRFR demand load are provided in Table 6-15 and Table 6-16.

Table 6-13. Summary of LFR demand bending moments and shear forces in the exterior beam due to Type 3-3 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-167.51	-92.63	37.27	58.06
Shear Force (kip)	17.50	13.93	10.79	6.37

Table 6-14. Summary of LFR demand bending moments and shear forces in the interior beam due to Type 3-3 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-284.04	-157.07	63.19	98.46
Shear Force (kip)	29.67	23.62	18.30	10.80

Table 6-15. Summary of LRFR demand bending moments and shear forces in the exterior beam due to Type 3-3 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-174.06	-96.25	38.72	60.33
Shear Force (kip)	18.18	14.48	11.21	6.62

Table 6-16. Summary of LRFR demand bending moments and shear forces in the interior beam due to Type 3-3 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-295.14	-163.20	65.66	102.31
Shear Force (kip)	30.83	24.55	19.02	11.23

Design tandem

Complete results for LFR demand load at the four sections of the exterior and interior beams are provided in Table 6-17 and Table 6-18, respectively. Complete results for LRFR demand load are provided in Table 6-19 and Table 6-20.

Table 6-17. Summary of LFR demand bending moments and shear forces in the exterior beam due to tandem loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-238.97	-140.55	63.06	97.17
Shear Force (kip)	20.35	19.43	16.93	10.63

Table 6-18. Summary of LFR demand bending moments and shear forces in the interior beam due to Tandem loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-405.21	-238.32	106.94	164.76
Shear Force (kip)	34.51	32.94	28.70	18.02

Table 6-19. Summary of LRFR demand bending moments and shear forces in the exterior beam due to tandem loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-248.30	-146.04	65.53	100.96
Shear Force (kip)	21.15	20.19	17.59	11.04

Table 6-20. Summary of LRFR demand bending moments and shear forces in the interior beam due to tandem loading.

	Location along the beam			
Load	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-421.03	-247.63	111.11	171.20
Shear Force (kip)	35.86	34.23	29.82	18.73

HL93

The controlling LRFR HL93 (3.6.1.2 LRFD) live load shear for the continuous five supported spans is the design truck in combination with a uniform lane loading w_{HL93} (without impact) as shown in Appendix A (Figure A6). (For longer simple spans, the HL93 truck will control, as shown in Figure A5).

Complete results for LRFR demand load at the four sections of the exterior and interior beams are provided in Table 6-21 and Table 6-22, respectively.

Table 6-21. Summary of LRFR demand bending moments and shear forces in the exterior beam due to HL93 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-399.22	-212.14	79.94	127.66
Shear Force (kip)	38.07	30.02	22.16	11.82

Table 6-22. Summary of LRFR demand bending moments and shear forces in the interior beam due to HL93 loading.

Load	Location along the beam			
	Edge	1/3 Mid-span	2/3 Mid-span	Mid-span
Bending Moment (kip-ft)	-676.93	-359.71	135.55	216.46
Shear Force (kip)	64.56	50.91	37.59	20.03

7 Nominal Resistance of Sections

Nominal moment capacity, M_n, and nominal shear capacity, V_n

Since the exterior arch beam and the interior beam have different geometric properties, it was necessary to calculate the nominal capacity for both of them to provide a complete explanation of the behavior of the entire bridge.

Moment and shear capacity calculations for the exterior beam at the edge and mid-span sections are shown below. Capacities for all sections of the exterior and interior beams are listed in the tables in this chapter.

Square bars are shown in the drawings and are used in all calculations. Due to the age of the structure, this is probably a correct representation of the reinforcing steel.

Both AASHTO and the LRFD use the same methods to calculate nominal section moment capacity. In AASHTO, design flexural capacity is the product of the nominal flexural capacity and a strength reduction factor, φ, of 0.9. In the LRFD, the factored flexural resistance for the strength limit state is the product of the nominal flexural resistance and the resistance factor, φ, which is 0.9 for flexure. Therefore, only a single calculation is performed for each section to determine the flexural capacity for use in the load ratings. However, shear capacities are calculated differently in the two standards, and both methods are shown in the shear capacity calculations in order to calculate a correct shear capacity for each load rating method.

Exterior beam

Moment capacity at the edge of the exterior beam (negative moment)

Assume that the bottom layer of three No. 8 reinforcement steel bars carries compression and the rest of the reinforcement steel carries all tension, as shown in Figure 7-1.

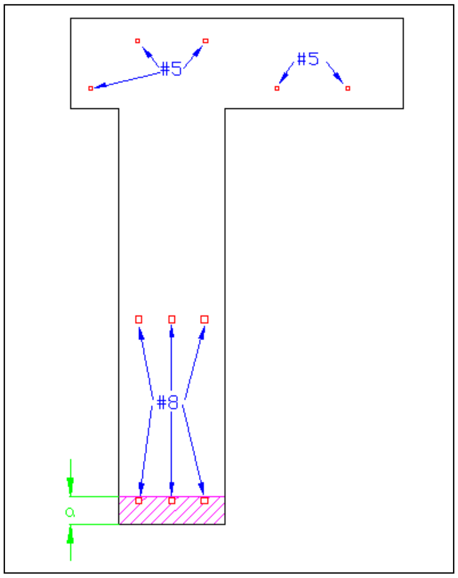


Figure 7-1. Cross section of the exterior beam at end-span.

Check if section is composite:

$$A_s \leq \frac{0.85 \times f'_c \times b_e \times t}{f_y}$$

$$A_s = A_{s1} + A_{s2} + A_{s3} = 2\#5 + 3\#5 + 3\#8$$

where area of the reinforcement steel is

$$A_{s\#5} = \left(\frac{5}{8}\right) in \times \left(\frac{5}{8}\right) in = 0.3906\, in^2$$

$$A_{s\#8} = \left(\frac{8}{8}\right) in \times \left(\frac{8}{8}\right) in = 1.0\, in^2$$

$$A_{s1} = 2 \times 0.3906\, in^2 = 0.7812\, in^2$$

$$A_{s2} = 3 \times 0.3906\, in^2 = 1.1718\, in^2$$

$$A_{s3} = 3 \times 1 in^2 = 3 in^2$$

$$A_s = 0.7812\, in^2 + 1.1718\, in^2 + 3 in^2 = 4.953\, in^2$$

Compression concrete area on the bottom web of the beam is

$$\frac{0.85\times f'_c\times b_e\times t}{f_y}=\frac{0.85\times 2.5\dfrac{kips}{in^2}\times 18\,in\times 15\,in}{33\dfrac{kips}{in^2}}=17.3864\,in^2$$

Since $A_s<\dfrac{0.85\times f'_c\times b_e\times t}{f_y}$, the section is composite; therefore, rectangular beam formulas are now valid.

Thus, tension on the top reinforcement is

$$T=A_s\times f_y=4.953\,in^2\times 33\frac{kips}{in^2}=163.449\,kips$$

$$C=T\Rightarrow 0.85\times f'_c\times b_e\times a$$

The depth of the compression block is

$$a=\frac{T}{0.85\times f'_c\times b_e}\qquad \text{(AASHTO 8-17 and LRFD 5.7.3.1.2)}$$

$$a=\frac{163.449\,kips}{0.85\times 2.5\dfrac{kips}{in^2}\times 18\,in}=4.2732\,in>3.5\,in$$

The depth of the neutral axis is

$$x=\frac{a}{\beta_1}\Rightarrow \beta_1=0.85\ \ if\ \ f'_c\le 4\,ksi\Rightarrow x=\frac{4.2732\ in}{0.85}=5.0273\,in$$

The nominal flexural strength of the cast-in-place arch beam is

$$M_n^-=A_s\times f_y\times\left[d-\left(\frac{a}{2}\right)\right]\qquad \text{(AASHTO 8-16 and LRFD 5.7.3.2.2)}$$

Distance between the tension and compression force (Figure 7-2):

$$d = h - \bar{y}$$

Centroid of the tensile steel reinforcement measured from top of the slab:

$$\bar{y} = \frac{2 \times A_{s\#5} \times d_1 + 3 \times A_{s\#5} \times d_2 + 3 \times A_{s\#8} \times d_3}{2 \times A_{s\#5} + 3 \times A_{s\#5} + 3 \times A_{s\#8}}$$

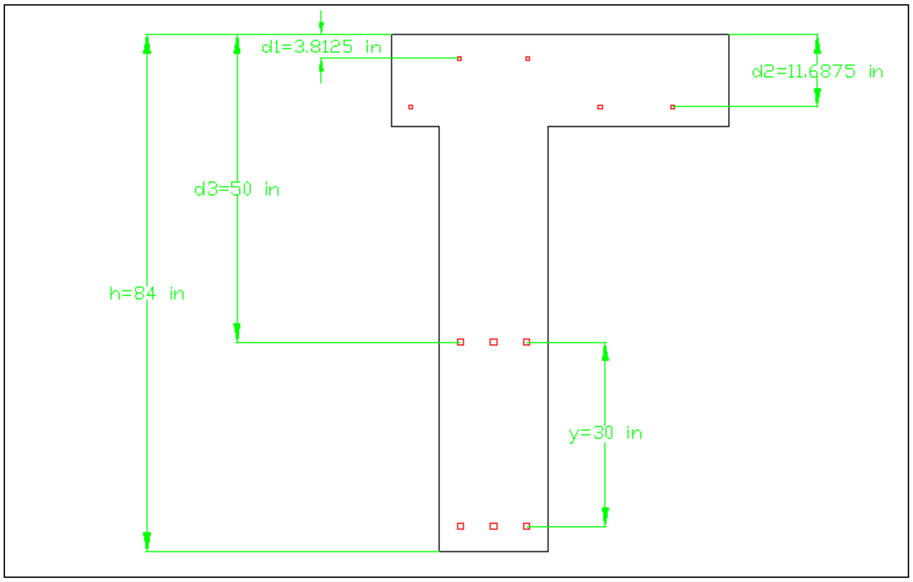


Figure 7-2. Distances of the steel rebar measure from the slab of the exterior beam.

where:

$$d_1 = Cover\ of\ the\ top\ of\ the\ slab + \frac{1}{2} \times height\ of\ a\ bar\ \#5$$

$$d_1 = 3.5in + \frac{1}{2} \times \left(\frac{5}{8}\right) in = 3.8125 in$$

$$d_2 = t_s - Cover\ of\ the\ bottom\ of\ the\ slab - \frac{1}{2} \times height\ of\ a\ bar\ \#5$$

$$d_2 = 15in - 3in - \frac{1}{2} \times \left(\frac{5}{8}\right) in = 11.6875 in$$

$$d_3 = h - \left(Cover\ of\ the\ bottom\ of\ the\ slab + \frac{1}{2} \times height\ of\ a\ bar\ \#8 + y \right)$$

$$d_3 = 84 in - \left(3.5 in + \frac{1}{2} \times \left(\frac{8}{8}\right) in + 30 in \right) = 50 in$$

$$\bar{y} = \frac{2\times 0.3906\,in^2 \times 3.8125\,in + 3\times 0.3906\,in^2 \times 11.6875\,in + 3\times 1\,in^2 \times 50\,in}{2\times 0.3906\,in^2 + 3\times 0.3906\,in^2 + 3\times 1\,in^2} = 33.6511\,in$$

$$d = 84\,in - 33.6511\,in = 50.3489\,in$$

The nominal flexural strength of the cast-in-place arch beam is

$$M_n^- = 4.953\,in^2 \times 33\frac{kips}{in^2} \times \left[50.3489\,in - \left(\frac{4.2732\,in}{2}\right)\right] \times \frac{1\,ft}{12\,in} = 656.6877\,k-ft$$

Verification of the tensile strain is calculated as

$$\varepsilon_t = \varepsilon_c \times \frac{d_t - x}{x}$$

where:

$$\varepsilon_c = Ultimate\,Strain\,of\,Concrete = 0.003$$

$$d_t = Effective\,Depth\,of\,the\,Extreme\,Tension\,Steel$$

$$d_t = h - \left(Cover\,of\,the\,bottom\,of\,the\,beam + \frac{1}{2} \times height\,of\,a\,bar\,\#5\right)$$

$$d_t = 84\,in - \left[3.5\,in + \frac{1}{2} \times \left(\frac{5}{8}\right) in\right] = 80.1875\,in$$

$$\varepsilon_t = 0.003 \times \frac{80.1875\,in - 5.0273\,in}{5.0273\,in} = 0.04485\frac{in}{in} > 0.005\frac{in}{in}, therefore$$

Since the net tensile strain is greater than 0.005, it is a tension-controlled section, and the strength reduction factor in flexure and shear is

$\phi = 0.9$ (AASHTO 8.16.1.2.2 and LRFD 6.5.4.2)

The capacity used in the LFR is the ultimate, or factored, capacity, while the nominal capacity is required for the LRFR. Therefore,

$$\phi \times Mn = 0.9 \times 656.6877\,k-ft = 591.0189\,k-ft$$

M_u = 591.02 k-ft (LFR capacity)

M_n = 656.69 *k-ft* (LRFR nominal capacity)

AASHTO shear capacity at the edge section of the exterior beam

$$\phi \times V_n = \phi \times (V_c + V_s)$$ (AASHTO 8-46 and 8-47)

$$V_c = 2 \times \sqrt{f'_c} \times b_w \times d$$ (AASHTO 8-51)

$$V_s = \frac{A_v \times f_y \times d}{s}$$ (AASHTO 9-30)

where:

$$\phi = 0.85$$ (AASHTO 8.16.1.2.2)

b_w = web width

Effective shear depth:

$$d = h - \overline{y}$$

From the previous calculation:

$$d = 50.3489 in$$

$$V_c = 2 \times \sqrt{2500 \frac{lb}{in^2}} \times 18 in \times 50.3489 in \times \frac{1 kip}{1000 lb} = 90.628 kips$$

No. 4 shear stirrups at 29-in. spacing are provided along the beam.

$$A_v = 2 \times A_{s\#4}$$

$$A_{s\#4} = \left(\frac{4}{8}\right) in \times \left(\frac{4}{8}\right) in = 0.25 in^2$$

$$A_v = 2 \times 0.25 in^2 = 0.50 in^2$$

$$V_s = \frac{0.50 in^2 \times 33 \frac{kips}{in^2} \times 50.3489 in}{29 in} = 28.65 kips$$

$$V_n = (90.628kips + 28.65kips) = 119.28kips$$

AASHTO design shear capacity for the LFR is given by

$$V_u = \phi \times V_n = 0.85 \times (90.628kips + 28.65kips) = 101.38kips$$

LRFD shear capacity at the edge section of the exterior beam

Nominal shear resistance is given as

$$V_n = (V_c + V_s)$$

for which

$$V_c = 0.0316 \times \beta \times \sqrt{f'_c} \times b_v \times d_v \qquad \text{(LRFD 5-68)}$$

$$V_s = \frac{A_v \times f_y \times d_v \times \cot\theta}{s} \qquad \text{(LRFD 5-69)}$$

$$\beta = 2.0 \qquad \text{(LRFD 5.8.3.4)}$$

$$\theta = 45^{\circ} \qquad \text{(LRFD 5.8.3.4)}$$

$$V_c = 0.0316 \times 2 \times \sqrt{2.5ksi} \times 18in \times 50.3489in = 90.56kips$$

No. 4 shear stirrups at 29-in. spacing are provided along the beam.

$$V_s = \frac{0.50in^2 \times 33ksi \times 50.3489in \times \cot(45)}{29in} = 28.64kips$$

LRFD design shear capacity is given by

$$V_n = (90.56kips + 28.64kips) = 119.27kips$$

Moment capacity at mid-span of the exterior beam (positive moment)

Assume that all the reinforcement steel carries all tension (Figure 7-3).

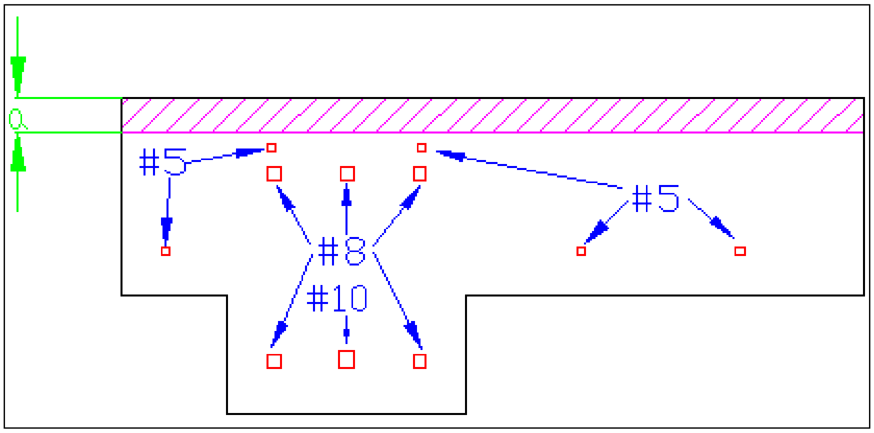


Figure 7-3. Cross section of the exterior beam at mid-span.

Check if section is composite:

$$A_s \le \frac{0.85 \times f'_c \times b_e \times t}{f_y}$$

Total steel area is

$$A_s = A_{s1} + A_{s2} + A_{s3} + A_{s4}$$
$$A_s = (2\#8 + 1\#10) + 3\#8 + 3\#5 + 2\#5$$

where:

$$A_{s\#5} = \left(\frac{5}{8}\right) in \times \left(\frac{5}{8}\right) in = 0.3906\, in^2$$

$$A_{s\#8} = \left(\frac{8}{8}\right) in \times \left(\frac{8}{8}\right) in = 1.0\, in^2$$

$$A_{s\#10} = \left(\frac{10}{8}\right) in \times \left(\frac{10}{8}\right) in = 1.5625\, in^2$$

$$A_{s1} = 2 \times 1 in^2 + 1 \times 1.5625 in^2 = 3.5625 in^2$$

$$A_{s2} = 3 \times 1 in^2 = 3 in^2$$

$$A_{s3} = 3 \times 0.3906 in^2 = 1.1718 in^2$$

$$A_{s4} = 2 \times 0.3906\,in^2 = 0.7812\,in^2$$

The total area of steel from the different layers is

$$A_s = 3.5625\,in^2 + 3\,in^2 + 1.1718\,in^2 + 0.7812\,in^2 = 8.5155\,in^2$$

Compression concrete area on the top flange of the beam is

$$\frac{0.85 \times f'_c \times b_e \times t}{f_y} = \frac{0.85 \times 2.5\dfrac{kips}{in^2} \times 56\,in \times 15\,in}{33\dfrac{kips}{in^2}} = 54.0909\,in^2$$

Since $A_s < \dfrac{0.85 \times f'_c \times b_e \times t}{f_y}$, the section is composite; therefore, rectangular beam formulas are now valid.

Tension on the steel is

$$T = A_s \times f_y = 8.5155\,in^2 \times 33\frac{kips}{in^2} = 281.0115\,kips$$

$$C = T \Rightarrow 0.85 \times f'_c \times b_e \times a = T$$

$$a = \frac{T}{0.85 \times f'_c \times b_e} = \frac{281.0115\,kips}{0.85 \times 2.5\dfrac{kips}{in^2} \times 56\,in} = 2.3614\,in < 3.5\,in$$

Therefore, all the reinforcement steel carries tension.

The assumption of all the reinforcement steel carrying all tension was correct.

Distance from the top flange to the neutral axis (Figure 7-4) is given by

$$x = \frac{a}{\beta_1} \Rightarrow \beta_1 = 0.85 \;\; if \;\; f'_c \le 4\,ksi \Rightarrow x = \frac{2.3614\;in}{0.85} = 2.7781\,in$$

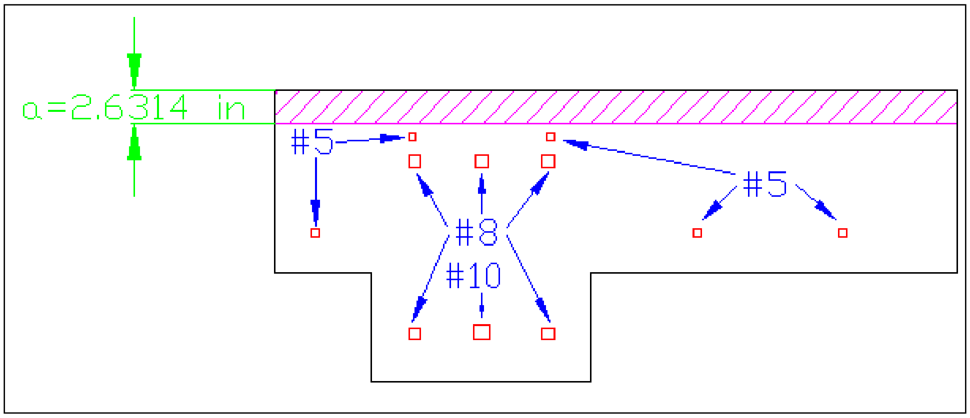


Figure 7-4. Cross section of the exterior beam at mid-span.

The equation to calculate the nominal bending capacity for the mid-span of the section is

$$M_n^+ = A_s \times f_y \times \left[d - \left(\frac{a}{2} \right) \right] \Rightarrow d = h - \bar{y}$$

$$\bar{y} = \frac{2 \times A_{s\#8} \times d_1 + A_{s\#10} \times d_2 + 3 \times A_{s\#8} \times d_3 + 3 \times A_{s\#5} \times d_4 + 2 \times A_{s\#5} \times d_5}{2 \times A_{s\#8} + A_{s\#10} + 3 \times A_{s\#8} + 3 \times A_{s\#5} + 2 \times A_{s\#5}}$$

The distance of each steel layer measure from the bottom of the exterior beam (Figure 7-5) is

$$d_1 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8$$

$$d_1 = 3.5\,in + \frac{1}{2} \times \left(\frac{8}{8} \right) in = 4.0000\,in$$

$$d_2 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#10$$

$$d_2 = 3.5\,in + \frac{1}{2} \times \left(\frac{10}{8} \right) in = 4.1250\,in$$

$$d_3 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8 + y$$

$$d_3 = 3.5\,in + \frac{1}{2} \times \left(\frac{8}{8} \right) in + 14.16\,in = 18.16\,in$$

$$d_4 = h - (t_s - Cover\ of\ the\ bottom\ of\ the\ slab) + \frac{1}{2} \times height\ of\ a\ bar\ \#5$$

$$d_4 = 24\,in - (15\,in - 3\,in) + \frac{1}{2} \times \left(\frac{5}{8}\right) in = 12.3125\,in$$

$$d_5 = h - Cover\ of\ the\ top\ of\ the\ slab - \frac{1}{2} \times height\ of\ a\ bar\ \#5$$

$$d_5 = 24\,in - 3.5\,in - \frac{1}{2} \times \left(\frac{5}{8}\right) in = 20.1875\,in$$

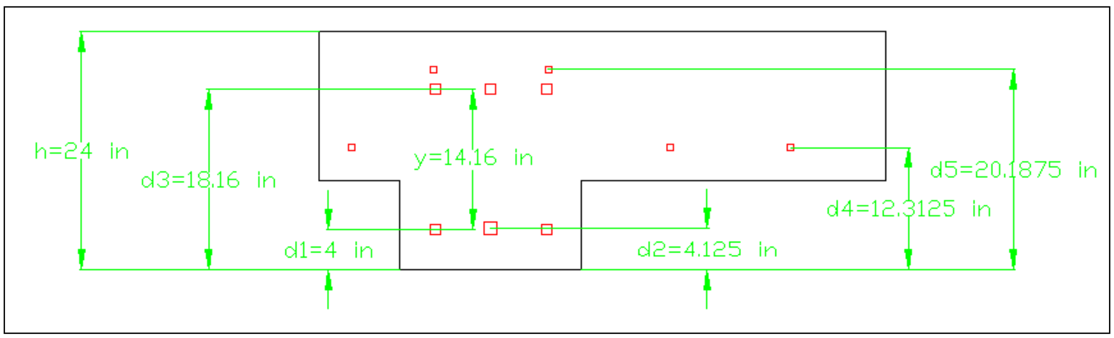


Figure 7-5. Centroid of the reinforcement steel to calculate the positive flexural capacity.

The centroid distance of the tension steel bars measured from the bottom of the exterior web is

$$\bar{y} = \frac{2 \times 1\,in^2 \times 4.0\,in + 1.5625\,in^2 \times 4.125\,in + 3 \times 1\,in^2 \times 18.16\,in + 3 \times 0.3906\,in^2 \times 12.3125\,in + 2 \times 0.3906\,in^2 \times 20.1875 \times in}{2 \times 1\,in^2 + 1.5625\,in^2 + 3 \times 1\,in^2 + 3 \times 0.3906\,in^2 + 2 \times 0.3906\,in^2} = 11.6404\,in$$

The distance between the compression force and the tension force is

$$d = 24\,in - 11.6404\,in = 12.3596\,in$$

The bending moment capacity for positive moment at the mid-span is

$$M_n^+ = 8.5155\,in^2 \times 33 \frac{kips}{in^2} \times \left[12.3596\,in - \left(\frac{2.3614\,in}{2}\right)\right] \times \frac{1\,ft}{12\,in} = 261.7833\,k - ft$$

Use the equation below to calculate the tensile strain on the tensile rebars (Figure 7-6):

$$\varepsilon_t = \varepsilon_c \times \frac{d_t - x}{x}$$

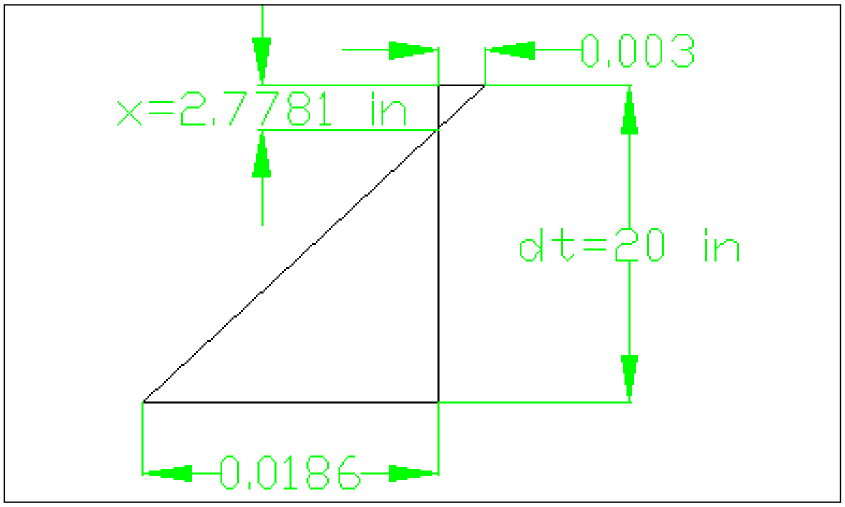


Figure 7-6. Strain diagram of the exterior beam at mid-span.

where:

$$\varepsilon_c = Ultimate\ \ Strain\ \ of\ \ \ Concrete = 0.003$$
$$d_t = Effective\ \ Depth\ \ of\ \ the\ \ Extreme\ Tension\ \ Steel$$
$$d_t = h - \left(Cover\ \ of\ \ the\ \ bottom\ \ of\ \ the\ \ beam + \frac{1}{2} \times height\ \ of\ a\ \ bar\ \#8 \right)$$
$$d_t = 24\,in - \left[3.5\,in + \frac{1}{2} \times \left(\frac{8}{8} \right) in \right] = 20\,in$$
$$\varepsilon_t = 0.003 \times \frac{20in - 2.7781in}{2.7781in} = 0.01860\frac{in}{in} > 0.005\frac{in}{in}, therefore$$

Since the tensile strain 0.01860 is higher than the limit set by LRFD code of 0.005, the reduction factor based on the failure mode is

$$Tension - controlled\ and\ \phi = 0.9 \qquad \text{(LRFD 6.5.4.2)}$$

Then the nominal bending moment capacity of the exterior arch beam is

$$\phi \times Mn = 0.9 \times 261.7833\,k - ft = 235.6050\,k - ft$$

Capacities for the load ratings are

M_u = 235.61 *k-ft* (LFR capacity)

M_n = 261.78 *k-ft* (LRFR capacity)

AASHTO shear capacity at the mid-span section of the exterior beam

$$\phi \times V_n = \phi \times (V_c + V_s) \qquad \text{(AASHTO 8-46 and 8-47)}$$

$$V_c = 2 \times \sqrt{f'_c} \times b_w \times d \qquad \text{(AASHTO 8-51)}$$

$$V_s = \frac{A_v \times f_y \times d}{s} \qquad \text{(AASHTO 9-30)}$$

where:

$$\phi = 0.85 \qquad \text{(AASHTO 8.16.1.2.2)}$$

Effective shear depth is given by

$$d = h - \overline{y}$$

$$d = 12.3596 in$$

$$V_c = 2 \times \sqrt{2500 \frac{lb}{in^2}} \times 18 in \times 12.3596 in \times \frac{1 kip}{1000 lb} = 22.25 kips$$

No. 4 shear stirrups at 29-in. spacing are provided along the beam.

$$A_v = 2 \times A_{s\#4}$$

$$A_{s\#4} = \left(\frac{4}{8}\right) in \times \left(\frac{4}{8}\right) in = 0.25 in^2$$

$$A_v = 2 \times 0.25 in^2 = 0.50 in^2$$

$$V_s = \frac{0.50 in^2 \times 33 \frac{kips}{in^2} \times 12.3596 in}{29 in} = 7.03 kips$$

AASHTO design shear capacity is given by

$$V_u = \phi \times V_n = 0.85 \times (22.25 kips + 7.03 kips) = 24.89 kips \qquad \text{(LFR capacity)}$$

LRFD shear capacity at the edge section of the exterior beam

Nominal shear resistance is given as

$$\phi \times V_n = \phi \times (V_c + V_s)$$

for which

$$V_c = 0.0316 \times \beta \times \sqrt{f'_c} \times b_v \times d_v \qquad \text{(LRFD 5-68)}$$

$$V_s = \frac{A_v \times f_y \times d_v \times \cot\theta}{s} \qquad \text{(LRFD 5-69)}$$

$$\beta = 2.0 \qquad \text{(LRFD 5.8.3.4)}$$

$$\theta = 45^\circ \qquad \text{(LRFD 5.8.3.4)}$$

$$V_c = 0.0316 \times 2 \times \sqrt{2.5ksi} \times 18in \times 12.3596in = 22.23kips$$

No. 4 shear stirrups at 29-in. spacing are provided along the beam.

$$V_s = \frac{0.50in^2 \times 33ksi \times 12.3596in \times \cot(45)}{29in} = 7.03kips$$

LRFD design shear capacity is given by

$$V_n = (22.23kips + 7.03kips) = 29.26kips$$

The procedure described above was used to calculate the nominal bending and shear capacities; Table 7-1 through Table 7-4 present a summary of the capacities. The difference between the AASHTO and LRFD capacities is the reduction factor. Since the bridge is in poor condition, the reduction factor for the LRFD method is 0.85 as calculated with the equation. Examples of the capacity calculations are shown in Appendix C.

Figure 5-8 shows the different locations considered in this study. The bending moment and shear forces where calculated every one-third (at 1.5 ft, 9 ft, 16.67 ft and 25.7 ft) distances measured from one end of the beam thru the mid-span.

Summary of the nominal capacity of the arch beams calculated on the critical point of the beam.

Table 7-1. Exterior beam ultimate capacities (LFR).

Location	Bending Moment (kip-ft)	Shear Force (kip)
Edge = 1.5 ft	-591.02	101.3836
1/3 Mid-Span = 9 ft	-359.85	63.40892
2/3 Mid-Span = 16.67 ft	285.37	31.4124
Mid-Span = 25.7 ft	235.60	24.86822

Table 7-2. Exterior beam nominal capacities (LRFD).

Location	Bending Moment (kip-ft)	Shear Force (kip)
Edge = 1.5 ft	-656.689	119.2095
1/3 Mid-Span = 9 ft	-399.833	74.5579
2/3 Mid-Span = 16.67 ft	317.0111	36.93564
Mid-Span = 25.7 ft	261.7778	29.24071

Table 7-3. Interior beam ultimate capacities (LFR).

Location	Bending Moment (kip-ft)	Shear Force (kip)
Edge = 1.5 ft	-1050.16	171.7239
1/3 Mid-Span = 9 ft	-653.56	109.5237
2/3 Mid-Span = 16.67 ft	826.67	64.56477
Mid-Span = 25.7 ft	604.52	48.76022

Table 7-4. Interior beam nominal capacities (LRFD).

Location	Bending Moment (kip-ft)	Shear Force (kip)
Edge = 1.5 ft	-1166.84	201.9024
1/3 Mid-Span = 9 ft	-726.178	128.7713
2/3 Mid-Span = 16.67 ft	918.5222	75.91131
Mid-Span = 25.7 ft	671.6889	57.32929

8 Load Rating Calculations

Load factor rating (LFR)

MCEB (Section 6.5) provides the load rating equation and the inventory and operating load factors for use in the LFR.

The MCEB defines the load rating factor for flexural and shear strength as

$$RF = \frac{C - A_1 D}{A_2 L(1+I)} \qquad \text{(MCEB 6.5.1)}$$

where:

$$C = Capacity\ of\ the\ beam$$
$$A_1 = Dead\ Load\ Factor \quad (1.3\ for\ Operating\ and\ Inventory)$$
$$D = Dead\ Load$$
$$A_2 = Live Load Factor \quad \begin{pmatrix} 1.3\ for\ Operating; \\ 2.17\ for\ Inventory \end{pmatrix}$$
$$L = Live\ Load$$

The **inventory rating level** corresponds to customary design-type loads while reflecting the existing condition of the structure. For calculations based on force and moment, the current condition of the bridge is considered in the capacity of the section used in the calculation.

The **operating rating level** corresponds to the maximum permissible live load the structure can safely withstand.

Further, the inventory load rating accommodates live loads that a bridge can carry for an indefinite period, while the operating load rating refers to live loads that could potentially shorten the bridge life if applied on a routine basis.

The rating of the bridge in tons is

$$RT = RF \times W \qquad \text{(MCEB 6.5.1)}$$

where W is the weight of the truck used to derive the demand live loading.

Presented below is an example of the rating factor for the exterior arch beam, considering the design vehicle HS20-44.

Negative bending moment at the support

Capacities for the exterior beam calculated using AASHTO are found in Table 7-1.

$$C = \phi \times M_n^- = 591.0189\, k - ft$$

Dead loads calculated in the DL + SDL FE analysis are found in Table 5-1.

$$D = M_{DL}^- = 354.28\, k - ft$$

Demand live load for the HS20-44 loading at the exterior edge section is found in Table 6-1:

$$L = M_{LL}^- = 261.33 k - ft$$

For the Operating Level rating:

$$RF = \frac{591.02\, k - ft - (1.3 \times 354.28\, k - ft)}{1.3 \times 261.33\, k - ft} = 0.38$$

For the Inventory Level rating:

$$RF = \frac{591.02\, k - ft - (1.3 \times 354.28\, k - ft)}{2.17 \times 261.33\, k - ft} = 0.23$$

Shear force

From Table 7-1,

$$C = \phi \times V_n = 101.38k$$

From Table 5-1,

$$D = V_{DL} = 33.68k$$

From Table 6-1,

$$L = V_{LL} = 26.2929k\,,$$

For the Operating Level rating:

$$RF = \frac{101.38k - (1.3 \times 33.68k)}{1.3 \times 26.2929k}$$
$$RF = 1.68$$

For the Inventory Level rating:

$$RF = \frac{101.38k - (1.3 \times 33.68k)}{2.17 \times 26.2929k}$$
$$RF = 1.1$$

Positive bending moment at the mid-span

From Table 7-1,

$$C = \phi \times M_n^+ = 235.60k - ft$$

From Table 5-1,

$$D = M_{DL}^+ = 69.10k - ft\,,$$

From Table 6-1,

$$L = M_{LL}^- = 92.30k - ft\,,$$

For the Operating Level load rating:

$$RF = \frac{235.60k - ft - (1.3 \times 69.10k - ft)}{1.3 \times 92.30k - ft} = 1.21$$

For the Inventory Level load rating:

$$RF = \frac{235.605k - ft - (1.3 \times 69.12k - ft)}{2.17 \times 92.2988k - ft} = 0.73$$

The LFR load rating factors calculated with the design vehicle HS20-44 and the AASTHO legal loads vehicles are summarized in Table 8-1 through Table 8-4.

Note from Table 8-1 through Table 8-2 that flexure controls the load ratings in the exterior beam. The maximum operating and inventory load ratings for the beam are the minimum ratings from Table 8-1, which occur at the edge section.

The MCEB LFR load rating factors calculated with AASHTO legal loads are provided in Table 8-3 and Table 8-4. Flexure also controls the legal load ratings, and maximum operating and inventory ratings for the beam are the minimum ratings from Table 8-3, which occur at the edge section.

Results for the interior beam are summarized in the tables below. Flexure at the edge section also controls the load ratings for the interior beam. For additional information regarding the flexure and shear capacities of the interior arch beam see Appendix C.

The MCEB LFR load rating factors calculated with AASHTO legal loads are provided in Table 8-7 and Table 8-8. Controlling ratings are highlighted; they are for flexure at the edge section of both exterior and interior beams.

Table 8-1. Summary of MCEB LFR for exterior beam flexure due to the HS20-44 loading.

Rating Type	Flexure															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
Rating	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Inventory	591.02	0.23	36	8.28	359.84	0.47	36	16.92	285.37	1.69	36	60.84	235.60	0.73	36	26.28
Operating	591.02	0.38	36	13.68	359.84	0.78	36	28.08	285.37	2.83	36	103.68	235.60	1.21	36	43.56

Table 8-2. Summary of MCEB LFR for exterior beam shear due to the HS20-44 loading.

Rating Type	Shear Force															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
Rating	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Inventory	101.38	1.01	36	36.36	63.41	0.76	36	27.36	31.41	0.44	36	15.84	24.87	0.99	36	35.64
Operating	101.38	1.69	36	60.84	63.41	1.27	36	45.72	31.41	0.73	36	26.28	24.87	1.66	36	59.76

Table 8-3. Summary of MCEB LFR for exterior beam flexure due to AASHTO legal loads.

Rating Type	Flexure															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
Rating	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Type 3																
Inventory	591.02	0.31	25	7.75	359.84	0.60	25	15.00	285.37	2.51	25	62.75	235.60	0.95	25	23.75
Operating	591.02	0.52	25	13	359.84	1.00	25	25.00	285.37	4.18	25	104.50	235.60	1.59	25	39.75
Type 3S2																
Inventory	591.02	0.32	36	11.52	359.84	0.63	36	22.68	285.37	2.83	36	101.88	235.60	1.03	36	37.08
Operating	591.02	0.54	36	19.44	359.84	1.07	36	38.52	285.37	4.72	36	169.92	235.60	1.72	36	61.92
Type 3-3																
Inventory	591.02	0.36	40	14.40	359.84	0.72	40	28.80	285.37	2.88	40	115.20	235.60	1.16	40	46.40
Operating	591.02	0.60	40	24.00	359.84	1.21	40	48.40	285.37	4.81	40	192.40	235.60	1.93	40	77.20

Table 8-4. Summary of MCEB LFR for exterior beam shear due to the AASHTO legal loads.

Rating Type	Shear Force															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
Rating	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Type 3																
Inventory	101.38	1.39	25	34.75	63.41	1.02	25	25.50	31.41	0.57	25	14.25	24.86	1.37	25	34.25
Operating	101.38	2.32	25	58.00	63.41	1.70	25	42.50	31.41	0.95	25	23.75	24.86	2.29	25	57.25
Type 3S2																
Inventory	101.38	1.48	36	53.28	63.41	1.07	36	38.52	31.41	0.59	36	21.24	24.86	1.48	36	53.28
Operating	101.38	2.46	36	88.56	63.41	1.79	36	64.44	31.41	0.98	36	35.28	24.86	2.46	36	88.56
Type 3-3																
Inventory	101.38	1.52	40	60.80	63.41	1.18	40	47.20	31.41	0.68	40	27.20	24.86	1.63	40	65.20
Operating	101.38	2.53	40	101.20	63.41	1.96	40	78.40	31.41	1.14	40	45.60	24.86	2.73	40	109.20

Table 8-5. Summary of MCEB LFR for interior beam flexure due to the HS20-44 loading.

Rating Type	Flexure															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
Rating	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Inventory	1050.16	0.26	36	9.36	653.55	0.53	36	19.08	826.67	3.17	36	114.12	604.52	1.34	36	48.24
Operating	1050.16	0.43	36	15.48	653.55	0.88	36	31.68	826.67	5.30	36	190.80	604.52	2.24	36	80.64

Table 8-6. Summary of MCEB LFR for interior beam shear due to the HS20-44 loading.

Rating Type	Shear Force															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
Rating	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Inventory	171.73	0.98	36	35.28	109.52	0.77	36	27.72	64.56	0.62	36	22.32	48.76	1.15	36	41.40
Operating	171.73	1.63	36	58.68	109.52	1.28	36	46.08	64.56	1.03	36	37.08	48.76	1.92	36	69.12

Table 8-7. Summary of MCEB LFR for interior beam flexure due to AASHTO legal loads.

Rating Type	Flexure															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
Rating	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Type 3																
Inventory	1050.16	0.34	25	8.5	653.55	0.68	25	17.00	826.67	4.68	25	117.00	604.52	1.76	25	44.00
Operating	1050.16	0.58	25	14.50	653.55	1.13	25	28.25	826.67	7.82	25	195.50	604.52	2.93	25	73.25
Type 3S2																
Inventory	1050.16	0.36	36	12.96	653.55	0.72	36	25.92	826.67	5.29	36	190.44	604.52	1.89	36	68.04
Operating	1050.16	0.60	36	21.60	653.55	1.20	36	43.20	826.67	8.84	36	318.24	604.52	3.17	36	114.12
Type 3-3																
Inventory	1050.16	0.40	40	16.00	653.55	0.82	40	32.80	826.67	5.39	40	215.60	604.52	2.13	40	85.20
Operating	1050.16	0.67	40	26.80	653.55	1.36	40	54.40	826.67	9.00	40	360.00	604.52	3.56	40	142.40

Table 8-8. Summary of MCEB LFR for interior beam shear due to the AASHTO legal loads.

Rating Type	Shear Force															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
Rating	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
Type 3																
Inventory	171.73	1.35	25	33.75	109.52	1.03	25	25.75	64.56	0.80	25	20.00	51.62	1.59	25	39.75
Operating	171.73	2.25	25	56.25	109.52	1.73	25	43.25	64.56	1.34	25	33.50	51.62	2.65	25	66.25
Type 3S2																
Inventory	171.73	1.43	36	51.48	109.52	1.10	36	39.60	64.56	0.84	36	30.24	51.62	1.71	36	61.56
Operating	171.73	2.39	36	86.04	109.52	1.82	36	65.52	64.56	1.40	36	505.40	51.62	2.86	36	102.96
Type 3-3																
Inventory	171.73	1.47	40	58.80	109.52	1.19	40	47.60	64.56	0.96	40	38.40	51.62	1.90	40	76.40
Operating	171.73	2.46	40	98.40	109.52	2.00	40	80.00	64.56	1.61	40	64.40	51.62	3.16	40	126.40

Load resistance and factor rating (LRFR)

The general LRFR equation is

$$RF = \frac{C - \gamma_{DC} DC - \gamma_{DW} DW}{\gamma_L (LL + IM)} \qquad \text{(LRFR 6.4.2)}$$

where:

C = capacity of the controlling longitudinal girder

R_n = nominal member resistance

DC = dead load effect (structural members and attachments)

DW = dead load from bridge deck overlays and utilities

$LL+IM$ = live load influence including dynamic impact

γ_{DC} = LRFD load factor for structural components and attachments

γ_{DW} = LRFD load factor for deck overlays and utilities

γ_L = evaluation live load factor

Resistance factor (RF) is first calculated for a design load rating using the HL93 notional loading. If RF < 1, a legal load rating is performed to determine a bridge rating in tons:

$$RT = RF \times W \qquad \text{(LRFR 6.4.4.4, calculated only for legal loads)}$$

Where W is the weight of the truck used to derive the live loading $LL+IM$.

Note that calculating RT for the HL93 notional load is not required by the LRFR and may be misleading because it includes the influence of both a truck and lane loading.

Only one limit state, Strength I, was evaluated in the LRFR procedure. The Strength I limit state is the basic load combination for normal vehicular bridge use and is the limit state to be used for a legal load rating. The factors for the load case and reliability level for this limit state are summarized in Table 8-9. The Strength I factor for the legal load rating is determined by the average daily truck traffic (ADTT) which was reported as zero in the 2009 inspection report due to closure of the bridges. However, once the bridges are load rated, they may be opened for public access, and the expected ADTT is unknown.

Table 8-9. LRFR load factors for Strength I and Service II (LRFR Table 6.1).

Parameter	Dead Load DC	Dead Load DW	Design Rating Inventory LL	Design Rating Operating LL	Legal Load LL
Strength I	1.25	1.25[a]	1.75	1.35	1.80[b]

a Thickness is field verified.

b Using unknown ADTT since the bridge is currently closed to traffic but may be opened for unknown public use in the future.

The flexural capacity of the exterior and interior beams C is defined as

$$C = \varphi_c \varphi_s \varphi R_n \qquad \text{(LRFR 6.4.2 for strength limit states)}$$

where:

φ_c = condition factor

φ_s = system redundancy factor

φ = LRFD resistance factor

and

$\varphi_c \, \varphi_s \geq 0.85$.

Since the condition of the superstructure was found to be “Poor” in the bridge inspection,

$$\varphi_c = \mathbf{0.85} \qquad \text{(LRFR Table 6-2)}$$

φ_c accounts for the existing condition of the bridge. For bridges in “Poor” condition, φ_c = 0.85.

For a three-girder bridge with girder spacing greater than 6 ft,

$$\varphi_s = \mathbf{1.0} \qquad \text{(LRFR Table 6-3)}$$

For flexure in reinforced concrete and shear in normal weight concrete,

$$\varphi = \mathbf{0.9} \quad \text{(LRFD 6.5.4.2)}$$

φ is a strength reduction factor that takes into account the variability in strength caused by uncertainties in material properties and workmanship.

$\varphi_c \varphi_s \geq 0.85$, condition is satisfied.

Design load rating

The design load rating is performed at two levels of reliability which are consistent with the inventory and operating levels specified in the MCEB. Both rating levels are performed with the HL93 loading defined in Appendix A.

Strength I limit state

From Table 7-2, the nominal moment capacity of the exterior beam at the edge section is

$$C = \varphi_c \varphi_s \varphi M_n^- = 0.85 \times 1.0 \times 0.9 \times 656.689\, k - ft = 502.37 k - ft$$

The HL93 loading effect at the edge of the exterior beam, from Table 6-21 is

$$M_{HL-93}^- (I + DF) = 399.22\, k - ft$$

Example Calculation - LRFR Inventory Design Load Rating

From Table 5-1:

$$D = M_{DL}^- = 354.28 k - ft$$

From Table 6-21:

$$L = M_{LL}^- = 399.22\, k - ft$$

For the Operating Level rating:

$$RF = \frac{502.37\,k - ft - (1.25 \times 354.28\,k - ft)}{1.35 \times 399.22\,k - ft} = 0.11$$

For the Inventory Level rating:

$$RF = \frac{502.37\,k - ft - (1.25 \times 354.28\,k - ft)}{1.75 \times 399.22\,k - ft} = 0.09$$

Legal load rating

The LRFR legal load rating is conducted in the same manner as the design load rating except that the AASHTO Type 3 legal load (see Appendix A) is considered instead of the HL93 and the LL+IM factor is now based on the ADTT (for inventory level), and calculations are only performed at the operating load level. It should be noted that the legal load rating calculation is required according to the LRFR since the design operating rating factor is less than one for this bridge. Refer to LRFR Appendix A.6.1 for a flowchart of the LRFR load rating procedure (see also Figure A7).

Strength I limit state

The Type 3 legal load LL+IM is obtained from Table 6-7, and the load factor from only one limit state (Strength I) was evaluated in the LRFR procedure. The Strength I limit state is the basic load combination for normal vehicular bridge use and is the limit state to be used for a legal load rating. The factors for the load case and reliability level for this limit state are summarized in Table 8-9. The Strength I factor for the legal load rating is determined by the ADTT, which was reported as zero in the 2009 report, due to closure of the bridges. However, once the bridges are load rated, they may be opened for public access, and the expected ADTT is unknown.

$$RF = \frac{502.37\,k - ft - (1.25 \times 354.28\,k - ft)}{1.80 \times 201.56\,k - ft} = 0.16$$

Tables 8-10 through Table 8-13 summarize the LRFR design and legal load ratings for both exterior and interior beams. Controlling ratings are highlighted; they are for flexure at the edge section.

Due to the “poor” condition of the concrete on the bridge observed during the previous bridge inspection, the load rating is reduced by approximately 40 percent when using the LRFR method. This reduction corresponds to a “poor” condition, which occurs when the condition factor (φc) is equal to 0.85.

In Tables 8-10 through 8-13, a vehicle weight of 25 tons rather than 36 tons for the HL93 rating indicates that the controlling design load at the applicable section was the design tandem.

Table 8-10. LRFR summary for exterior beam flexure due to design (HL93) and legal loads.

Rating Type	Strength I Limit State - Flexure															
Section	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
Rating	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
HL93																
Inventory	502.37	0.09	36	3.24	305.23	0.27	36	9.72	242.51	1.38	25	34.50	200.26	0.51	25	12.75
Operating	502.37	0.11	36	3.69	305.23	0.35	36	12.60	242.51	1.78	25	44.50	200.26	0.66	25	16.50
Type 3 – Legal Load																
	502.37	0.16	25	4.00	305.23	0.48	25	12.00	242.51	2.40	25	60.00	222.52	0.87	25	21.75
Type 3S2– Legal Load																
	502.37	0.17	36	6.12	305.23	0.51	36	18.36	242.51	2.71	36	97.56	222.52	0.93	36	33.48
Type 3-3– Legal Load																
	502.37	0.19	40	7.60	305.23	0.57	40	22.80	242.51	2.76	40	110.40	222.52	1.05	40	42.00

Table 8-11. LRFR summary for exterior beam shear force due to design (HL93) and legal loads.

Rating	Strength I Limit State – Shear Force															
	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
HL93																
Inventory	91.20	0.74	36	26.64	57.04	0.58	36	20.88	28.25	0.35	25	8.75	22.37	0.98	25	24.50
Operating	91.20	0.96	36	34.56	57.04	0.75	36	27.00	28.25	0.45	25	11.25	22.37	1.27	25	31.75
Type 3 – Legal Load																
	91.20	1.37	25	34.25	57.04	1.00	25	25.00	28.25	0.45	25	11.25	22.37	1.42	25	35.50
Type 3S2– Legal Load																
	91.20	1.46	36	52.56	57.04	1.06	36	38.16	28.25	0.58	36	20.88	22.37	1.53	36	55.08
Type 3-3– Legal Load																
	91.20	1.50	40	60.00	57.04	1.16	40	46.40	28.25	0.66	40	26.40	22.37	1.70	40	68.00

Table 8-12. LRFR summary for interior beam flexure due to design (HL93) and legal loads.

Rating	Strength I Limit State - Flexure															
	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
HL93																
Inventory	892.63	0.10	36	3.60	555.53	0.31	36	11.16	702.67	2.61	25	65.25	513.84	0.98	25	24.50
Operating	892.63	0.13	36	4.68	555.53	0.40	36	14.40	702.67	3.38	25	84.50	513.84	1.27	25	31.75
Type 3 – Legal Load																
	892.63	0.19	25	4.75	555.53	0.55	25	13.75	702.67	4.55	25	113.75	513.84	1.66	25	41.50
Type 3S2– Legal Load																
	892.63	0.20	36	7.20	555.53	0.59	36	21.24	702.67	5.14	36	185.04	513.84	1.79	36	64.44
Type 3-3– Legal Load																
	892.63	0.22	40	8.80	555.53	0.66	40	26.40	702.67	5.24	40	209.60	513.84	2.01	40	80.40

Table 8-13. LRFR summary for interior beam shear force due to design (HL93) and legal loads.

Rating	Strength I limit state – Shear Force															
	Edge				1/3 Mid-Span				2/3 Mid-Span				Mid-Span			
	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT	C	RF	W	RT
	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons	kip-ft	-	tons	tons
HL93																
Inventory	154.46	0.71	36	25.56	98.51	0.58	36	20.88	58.07	0.50	25	12.50	43.86	1.13	25	28.25
Operating	154.46	0.92	36	33.12	98.51	0.76	36	27.36	58.07	0.65	25	16.25	43.86	1.47	25	36.75
Type 3 – Legal Load																
	154.46	1.33	25	33.25	98.51	1.02	25	25.50	58.07	0.80	25	20.00	43.86	1.65	25	41.25
Type 3S2– Legal Load																
	154.46	1.41	36	50.76	98.51	1.44	36	51.84	58.07	0.83	36	29.88	43.86	1.77	36	63.72
Type 3-3– Legal Load																
	154.46	1.45	40	58.00	98.51	1.57	40	62.80	58.07	0.96	40	38.40	43.86	1.96	40	78.40

9 Conclusions

This document presents two methods for load rating bridges, LFR and LRFR, with sample calculations and commentary. The load ratings performed in this study can either be defined as notional (HS20-44, HL93) or as legal load ratings for the vehicles shown in Appendix A.

The notional load rating procedures are often used as a first-tier approach (RF > 1 OK, RF < 1 needs more evaluation) consistent with a new bridge design, while the legal load ratings are a second-tier approach to evaluate the capacity of the bridge based on more realistic (and less conservative) highway loadings.

All notional rating factors are lower than one; therefore, legal load ratings were performed for both load rating methods. Controlling legal load ratings, shown in Chapter 8, were also less than vehicle loads, suggesting that AASHTO legal loads cannot be carried safely on the bridge and that a weight limit for each vehicle should be posted. Currently, the Lahontan Arch Spillway Bridge is closed because of the poor structural conditions visible on the previous bridge inspection. The RF results confirm this poor condition.

Table 9-1 shows a summary of the controlling ratings for the bridge for each of the applied loadings. Since the exterior beams represent the controlling element in capacity, all ratings in this table are for the edge section of the exterior beam. The relatively low amount of reinforcement near the support of each beam resulted in a low RF for negative moment on each beam. Because of this lower capacity, the bridge is unable to support the weight effects of the actual design vehicles, including the legal loads. This result is not unexpected, since the bridge was built in 1915 and AASHTO standards for truck loads were not published until 1935.

Several alternatives are available to improve the RF. An accurate assessment of ADTT based on historical usage could provide a more conservative value for the LRFR legal load factor. Nondestructive and destructive tests could provide accurate material properties and confirm rebar locations. Concrete coring is one method that could be used to provide actual rather than assumed properties for the concrete and steel rebars. Ground penetration radar could be used to identify the exact position of the principal reinforcement and eliminate uncertainties in the sections used to calculate capacities. A diagnostic load test could be performed on the bridge using a vehicle weight lower than the suggested postings to provide a better understanding of the real behavior of the bridge and a more precise load rating.

Table 9-1. Summary of MCEB and LRFR load rating results for the exterior beam.

Rating	Specification	Rating Type	Load Type	RF or RT (tons)		
				Inventory	Operating	Legal Load
Notional Rating	MCEB	LFD	HS20-44	0.23	0.38	—
	LRFR	Design	HL93	0.09	0.11	—
Legal Load Rating	MCEB	LFD	Type 3	7.75	13.00	—
	LRFR	Legal Load	Type 3	—	—	4.00
	MCEB	LFD	Type 3S2	11.52	19.44	—
	LRFR	Legal Load	Type 3S2	—	—	6.12
	MCEB	LFD	Type 3-3	14.40	24.00	—
	LRFR	Legal Load	Type 3-3	—	—	7.60

For the operating rating for the HS20-44 vehicle, the bridge rating is 0.38 × 36 tons, or 13 tons. This is a reasonable rating for a bridge that may have been designed for a loading lighter than a standard H-15 vehicle. To confirm or improve this rating, a diagnostic load test is strongly recommended.

For all ratings, the bridge was modeled as a composite section. Ratings are for the superstructure only. The substructure (piers and abutments) was assumed to be adequate to resist superstructure loadings. This is a typical assumption, since the substructure is normally designed to be stronger than the superstructure.

References

American Association of State Highway and Transportation Officials. Washington, DC.

1994. [**MCEB**] *AASHTO manual for the condition evaluation of bridges.* 2d ed.

2002. [**AASHTO**] *AASHTO standard specifications for highway bridges.* 17th ed.

2003. [**LRFR**] *AASHTO manual for condition evaluation and load and resistance factor rating (LRFR) of highway bridges.*

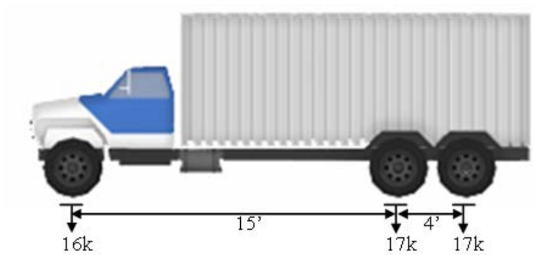
2004. [**LRFD**] *AASHTO LRFD bridge design specifications.* 3d ed.

Department of the Army. 2002. *Military, nonstandard fixed bridging*. Field Manual 3-34.343. Washington, DC.

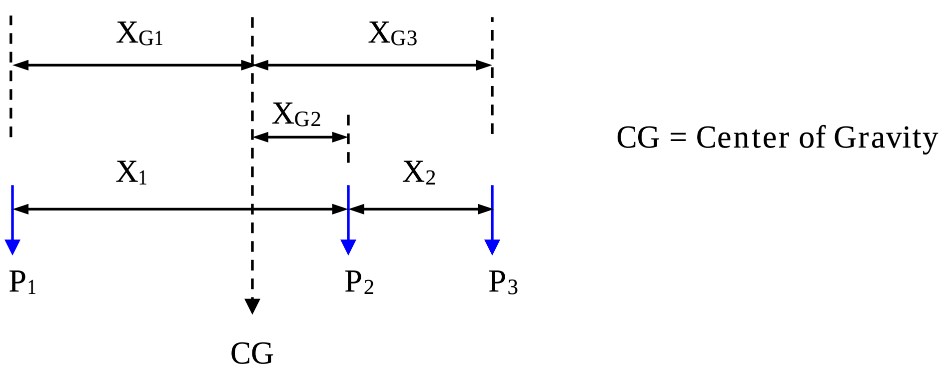
U.S. Army Engineer District, Sacramento. 2009. *Initial inventory bridge inspection report, Lahontan Dam Spillway bridges, Fallon, Nevada*. Sacramento, CA.

Appendix A: Live Load Descriptions

Standard AASHTO legal load and notional design vehicles will be referred to frequently in this report. The different configurations (Figures A1–A6) are defined as follows:



a. Typical Type 3 truck.

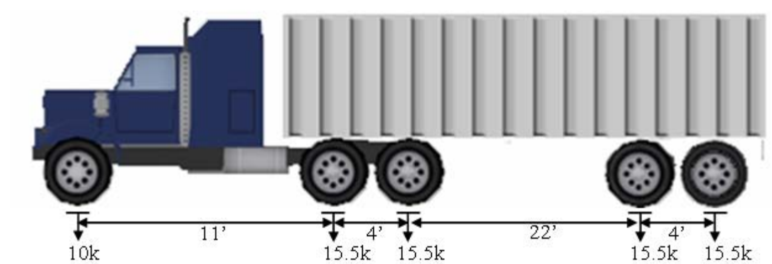


b. Loads and dimensions for use with Table A1.

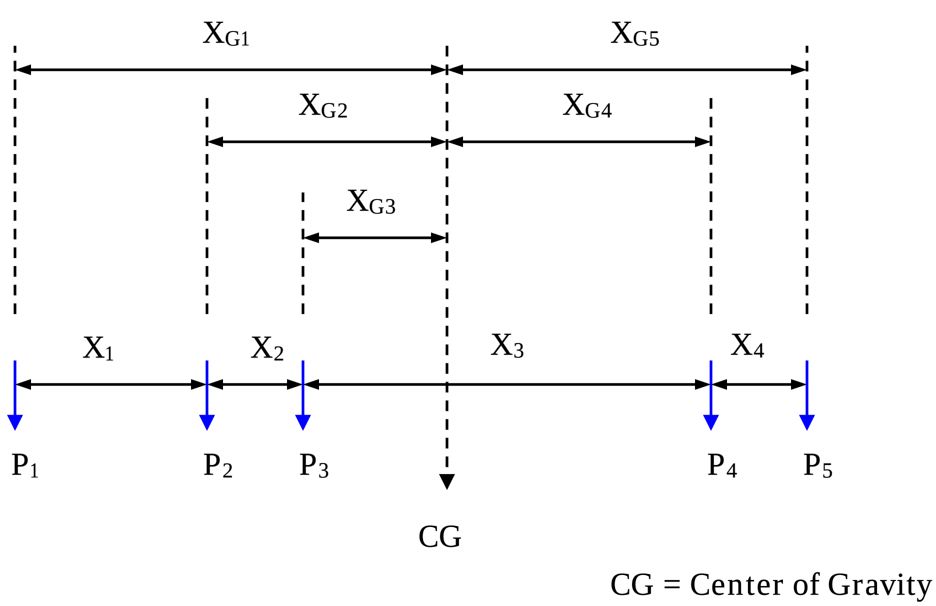
Figure A1. AASHTO Type 3.

Table A1. AASHTO Type 3 - loading and dimensions.

Loading Data – AASHTO Type 3 Total Weight = 50 kips (25 Tons)			
Axle Loads (k)	P_1	P_2	P_3
	16	17	17
Dimensions – AASHTO Type 3			
Longitudinal Spacing (ft)	X_1	X_2	
	15	4	
Distance to Center of Gravity (ft)	X_{G1}	X_{G2}	X_{G3}
	11.56	3.44	7.44



a. Typical Type 3S2 truck.

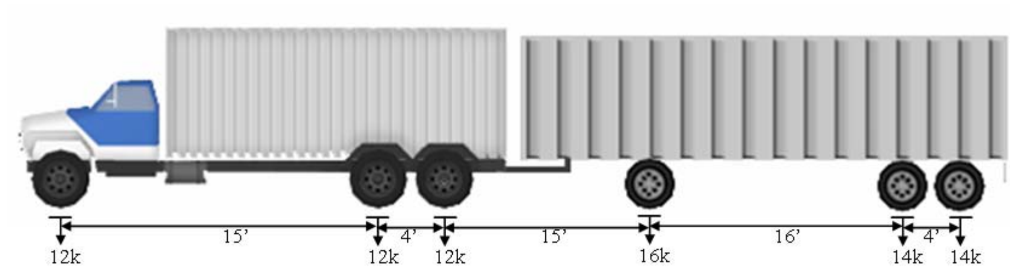


b. Loads and dimension for use with Table A2.

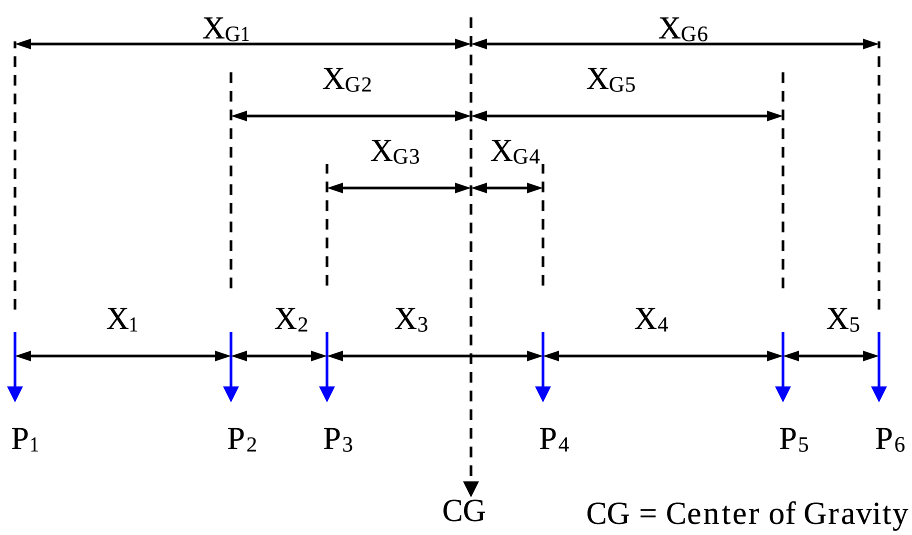
Figure A2. AASHTO Type 3S2.

Table A2. AASHTO Type 3S2 - loading and dimensions.

Loading Data – AASHTO Type 3S2 Total Weight = 72 kips (36 Tons)					
Axle Loads (k)	P_1	P_2	P_3	P_4	P_5
	10	15.5	15.5	15.5	15.5
Dimensions – AASHTO Type 3S2					
Longitudinal Spacing (ft)	X_1	X_2	X_3	X_4	
	11	4	22	4	
Distance to Center of Gravity (ft)	X_{G1}	X_{G2}	X_{G3}	X_{G4}	X_{G5}
	22.39	11.39	7.39	14.61	18.61



a, Typical Type 3-3 truck.

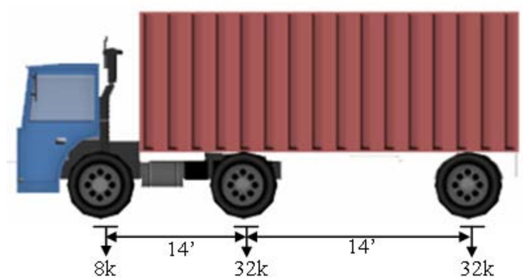


b. Loads and dimension for use with Table A3.

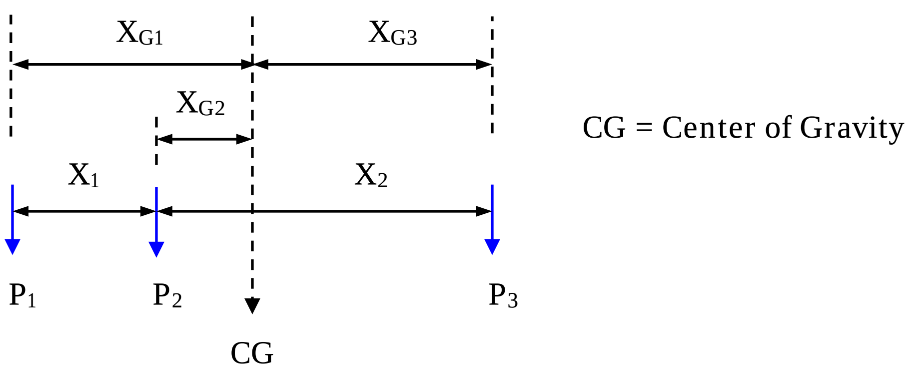
Figure A3. AASHTO Type 3-3.

Table A3. AASHTO Type 3-3 - loading and dimensions.

Loading Data – AASHTO Type 3-3 Total Weight = 80 kips (40 Tons)						
Axle Loads (k)	P_1	P_2	P_3	P_4	P_5	P_6
	12	12	12	16	14	14
Dimensions – AASHTO Type 3-3						
Longitudinal Spacing (ft)	X_1	X_2	X_3	X_4	X_5	
	15	4	15	16	4	
Distance to Center of Gravity (ft)	X_{G1}	X_{G2}	X_{G3}	X_{G4}	X_{G5}	X_{G6}
	30.1	15.1	11.1	3.9	19.9	23.9



a. Typical HS20-44 truck.

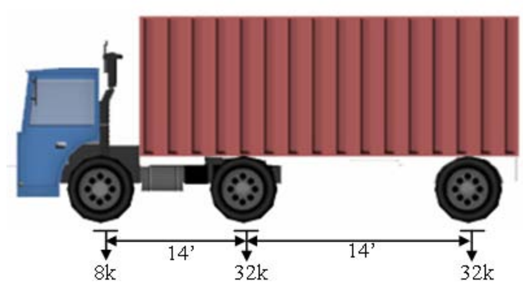


b. Loads and dimensions for use with Table A4.

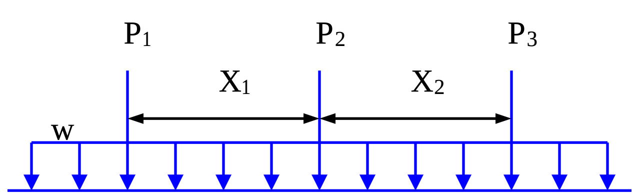
Figure A4. AASHTO notional vehicles: HS25-44, HS20-44, HS15-44 (1994).

Table A4. AASHTO HS25-44, HS20-44, and HS15-44 - loading and dimensions.

Loading Data – AASHTO HS20-44 and HS15-44 Total Weight HS25-44 = 90 kips (45 Tons) Total Weight: HS20-44 = 72 kips (36 Tons) Total Weight: HS15-44 = 54 kips (27 Tons)						
Axle Loads (k)	P_1		P_2		P_3	
HS25-44	10		40		40	
HS20-44	8		32		32	
HS15-44	6		24		24	
Dimensions – AASHTO HS20-44 and HS15-44						
Longitudinal Spacing (ft)	X_1		$X_{2\ MIN.}$		$X_{2\ MAX.}$	
HS25-44, HS20-44, and HS15-44	14		14		30	
Distance to Center of Gravity (ft)	Minimum			Maximum		
	X_{G1}	X_{G2}	X_{G3}	X_{G1}	X_{G2}	X_{G3}
HS25-44, HS20-44, and HS15-44	18.67	4.67	9.33	25.78	11.78	18.22



a. Typical design truck.

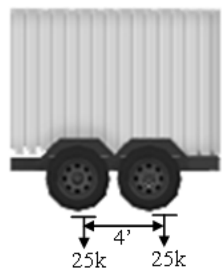


b. Loads and dimensions for use with Table A5.

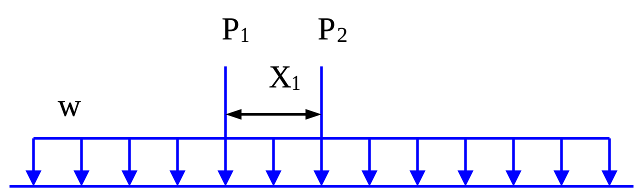
Figure A5. HL93 (design truck with lane load) (AASHTO 2003).

Table A5. HL93 (Design truck with lane load) - loading and dimensions (AASHTO 2003).

Loading Data – HL93 (Design Truck with Lane Load)			
Axle Loads (k)	P_1	P_2	P_3
	8	32	32
Uniform Lane Load (klf)	0.64		
Dimensions – HL93 (Design Truck with Lane Load)			
Longitudinal Spacing (ft)	X_1	X_2	
	14	14 to 30	



a. Typical design tandem.



b. Loads and dimensions for use with Table A6.

Figure A6. HL93 (design tandem with lane load) (AASHTO 2003).

Table A6. HL93 (Design truck with lane load) - loading and dimensions (AASHTO 2003).

Loading Data – HL93 (Design Tandem with Lane Load)		
Axle Loads (k)	P_1	P_2
	25	25
Uniform Lane Load (klf)	0.64	
Dimensions – HL93 (Design Tandem with Lane Load)		
Longitudinal Spacing (ft)	X_1	
	4	

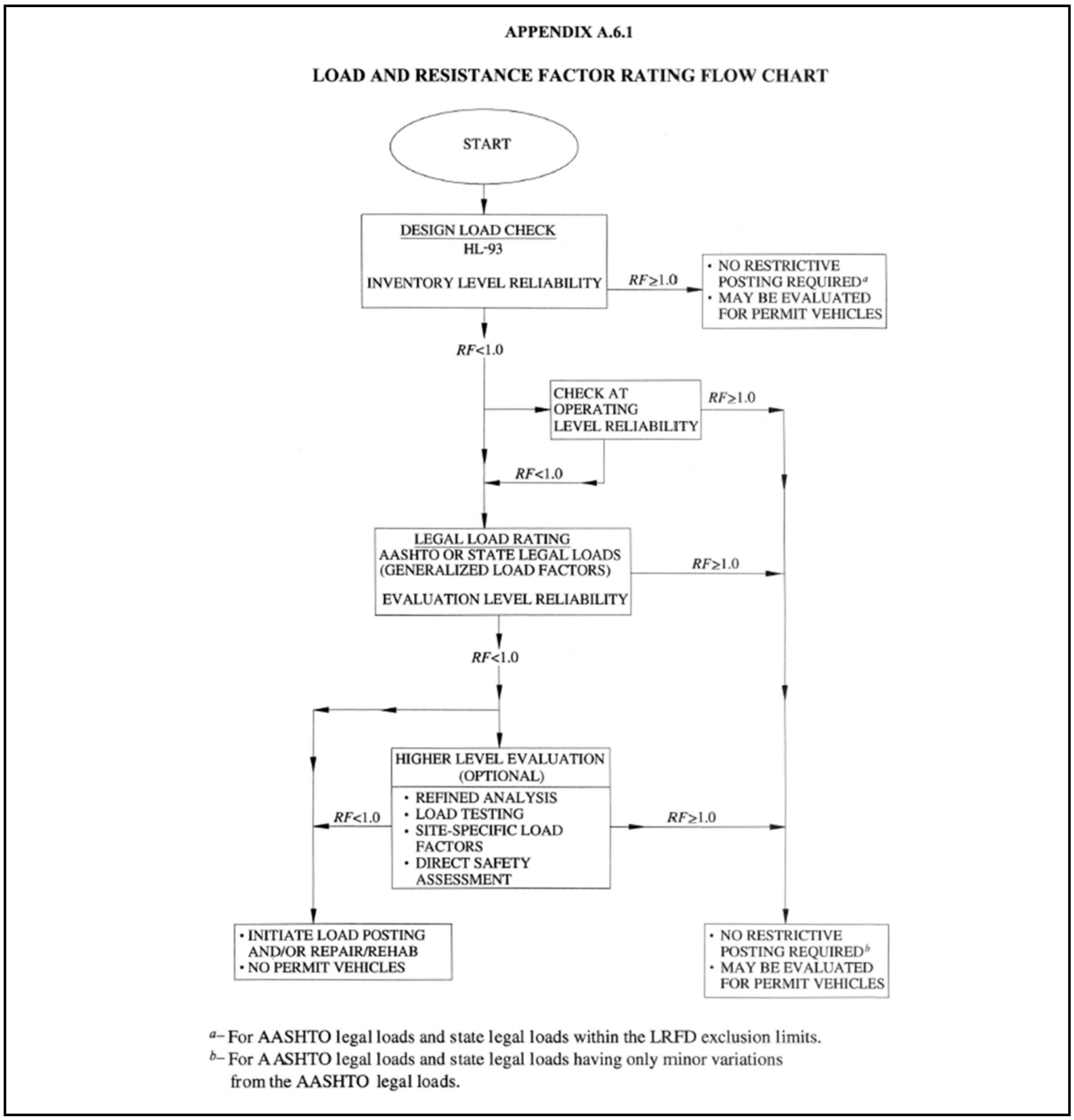


Figure A7. Flowchart of the LRFR load-rating procedure.

Appendix B: Bridge Railing Calculations

The weight of one bridge railing per foot was calculated using the drawing (Figure B1) provided on page 5316 of the November 1914 documentation from the U.S. Department of the Interior Reclamation Service.

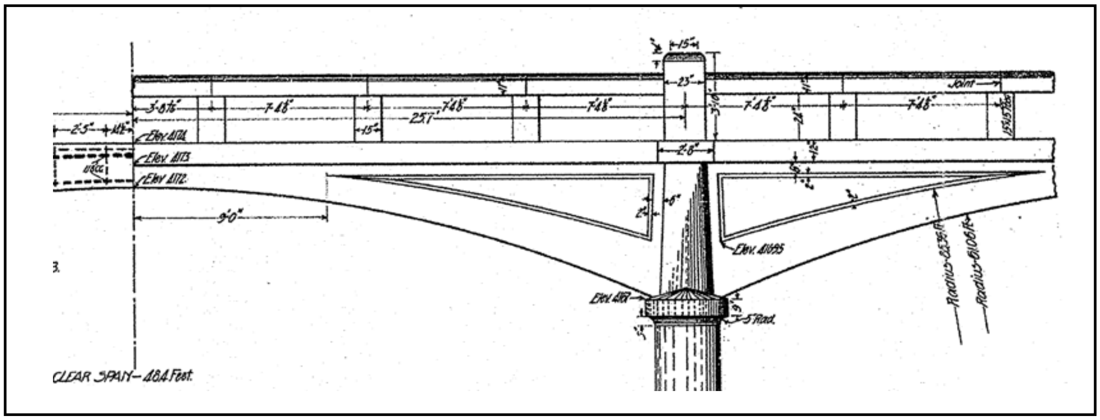


Figure B1. Bridge railing dimensions.

Each intermediate pier and abutment has two main posts, and each span has 12 intermediate posts, otherwise six per side.

$$W_{mainpost} = \frac{quantity \times V \times \gamma_{concrete}}{span\_length} = \frac{4 \times \left(\frac{46in \times 23in \times 23in}{12^3\, ft^3}\right) \times 0.15\frac{kips}{ft^3}}{3arch \times 51.4\, ft} = 0.055\frac{kips}{ft}$$

$$W_{post} = \frac{quantity \times V \times \gamma_{concrete}}{span\_length} = \frac{12 \times \left(\frac{33in \times 15in \times 15in}{12^3\, ft^3}\right) \times 0.15\frac{kips}{ft^3}}{3arch \times 51.4\, ft} = 0.051\frac{kips}{ft}$$

$$W_{rails} = quantity \times A \times \gamma_{concrete} = \frac{2}{3arch} \times \left(\frac{11in \times 15in}{12^2\, ft^2}\right) \times 0.15\frac{kips}{ft^3} = 0.12\frac{kips}{ft}$$

Total for railing (including an additional 10% for bolts and clips)

$$w_{SDL} = 1.10\left(W_{mainpost} + W_{post} + W_{rails}\right) = 1.10\left(0.055\frac{kips}{ft} + 0.051\frac{kips}{ft} + 0.12\frac{kips}{ft}\right) = 0.25\frac{kips}{ft}$$

Appendix C: Flexure and Shear Capacity,

Exterior beam

Nominal positive moment of the exterior beam at mid-span

Assume that all the reinforcement steel carries all tension (Figure C1).

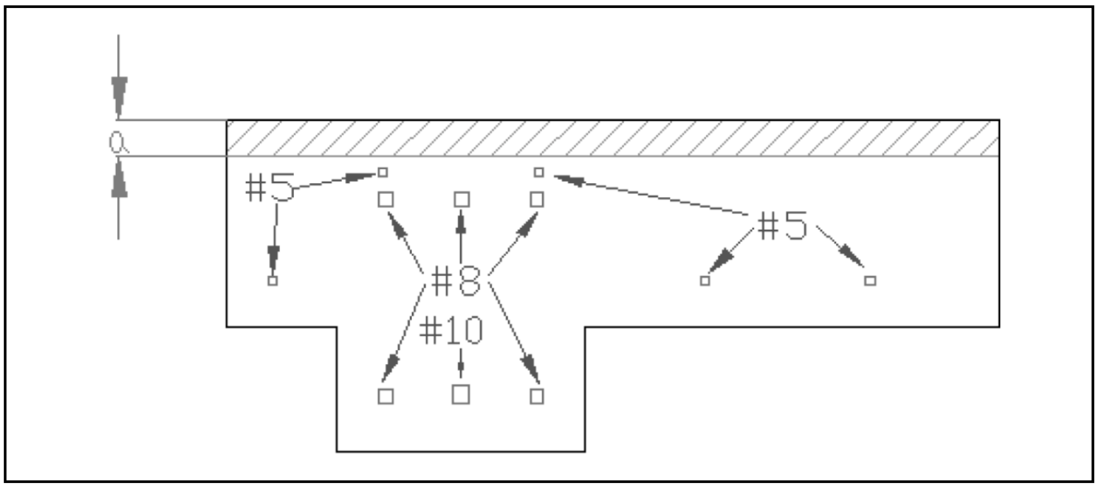


Figure C1. Cross section of the exterior beam at mid-span.

Assumption of all the reinforcement steel carrying all tension:

Check if section is composite:

$$A_s \le \frac{0.85 \times f'_c \times b_e \times t}{f_y}$$

$$A_s = A_{s1} + A_{s2} + A_{s3} + A_{s4} = (2\#8 + 1\#10) + 3\#8 + 3\#5 + 2\#5$$

where:

$$A_{s\#5} = \left(\frac{5}{8}\right) in \times \left(\frac{5}{8}\right) in = 0.3906\, in^2$$

$$A_{s\#8} = \left(\frac{8}{8}\right) in \times \left(\frac{8}{8}\right) in = 1.0\, in^2$$

$$A_{s\#10} = \left(\frac{10}{8}\right) in \times \left(\frac{10}{8}\right) in = 1.5625\, in^2$$

$$A_{s1} = 2 \times 1 in^2 + 1 \times 1.5625 in^2 = 3.5625\, in^2$$

$$A_{s2} = 3 \times 1 in^2 = 3 in^2$$

$$A_{s3} = 3 \times 0.3906\, in^2 = 1.1718\, in^2$$

$$A_{s4} = 2 \times 0.3906\, in^2 = 0.7812\, in^2$$

$$A_s = 3.5625\, in^2 + 3 in^2 + 1.1718\, in^2 + 0.7812\, in^2 = 8.5155\, in^2$$

$$\frac{0.85 \times f'_c \times b_e \times t}{f_y} = \frac{0.85 \times 2.5 \frac{kips}{in^2} \times 56\, in \times 15\, in}{33 \frac{kips}{in^2}} = 54.0909\, in^2$$

Since $A_s < \dfrac{0.85 \times f'_c \times b_e \times t}{f_y}$, the section is composite; rectangular beam formulas are now valid.

$$T = A_s \times f_y = 8.5155 in^2 \times 33 \frac{kips}{in^2} = 281.0115\, kips$$

$$C = T \Rightarrow 0.85 \times f'_c \times b_e \times a = T \Rightarrow a = \frac{T}{0.85 \times f'_c \times b_e} = \frac{281.0115\, kips}{0.85 \times 2.5 \frac{kips}{in^2} \times 56\, in} = 2.3614\, in < 3.5\, in$$

Therefore, all the reinforcement steel carries all tension.

Figure C2 shows that the assumption of all the reinforcement steel carrying all tension was correct.

$$x = \frac{a}{\beta_1} \Rightarrow \beta_1 = 0.85 \;\; if \;\; f'_c \leq 4\, ksi \Rightarrow x = \frac{2.3614\; in}{0.85} = 2.7781 in$$

$$M_n^+ = A_s \times f_y \times \left[d - \left(\frac{a}{2}\right) \right] \Rightarrow d = h - \overline{y}$$

$$\bar{y} = \frac{2 \times A_{s\#8} \times d_1 + A_{s\#10} \times d_2 + 3 \times A_{s\#8} \times d_3 + 3 \times A_{s\#5} \times d_4 + 2 \times A_{s\#5} \times d_5}{2 \times A_{s\#8} + A_{s\#10} + 3 \times A_{s\#8} + 3 \times A_{s\#5} + 2 \times A_{s\#5}}$$

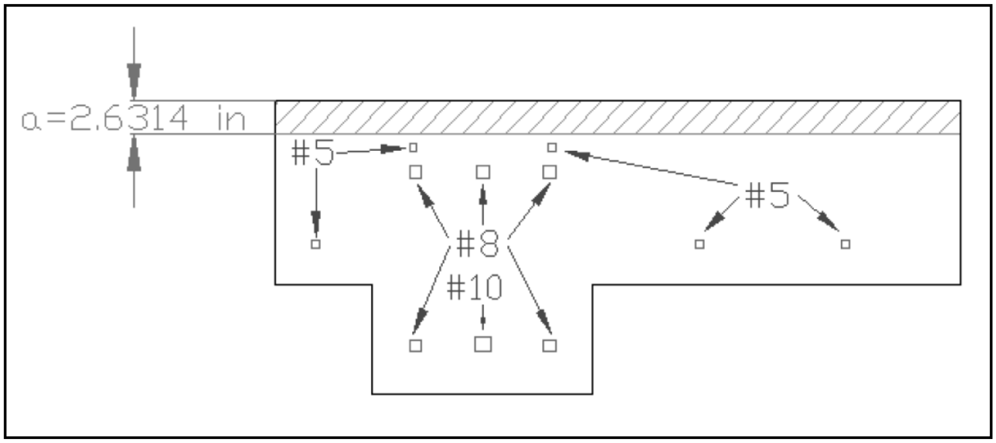


Figure C2. Cross section of the exterior beam at mid-span.

Figure C3 shows the distances that were needed to calculate the centroid of the reinforcement steel to calculate the positive flexural capacity. Figure C4 presents a strain diagram of the exterior beam at mid-span.

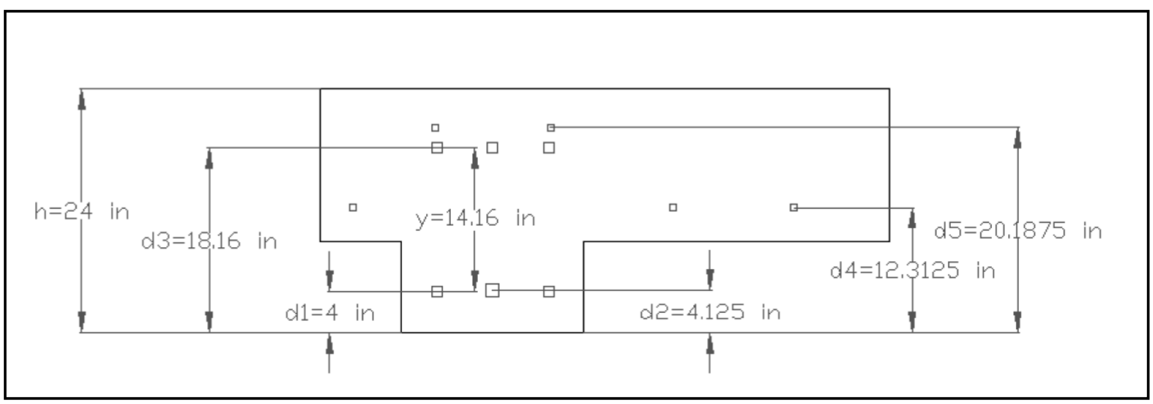


Figure C3. Cross section of the exterior beam at mid-span.

where:

$$d_1 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8 = 3.5\,in + \frac{1}{2} \times \left(\frac{8}{8}\right) in = 4.0000\,in$$

$$d_2 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#10 = 3.5\,in + \frac{1}{2} \times \left(\frac{10}{8}\right) in = 4.1250\,in$$

$$d_3 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8 + y$$

$$d_3 = 3.5\,in + \frac{1}{2} \times \left(\frac{8}{8}\right) in + 14.16\,in = 18.16\,in$$

$$d_4 = h - (t_s - Cover\ of\ the\ bottom\ of\ the\ slab) + \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 24\,in - (15\,in - 3\,in) +$$

$$\frac{1}{2} \times \left(\frac{5}{8}\right) in = 12.3125\,in$$

$$d_5 = h - Cover\ of\ the\ top\ of\ the\ slab - \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 24\,in - 3.5\,in - \frac{1}{2} \times \left(\frac{5}{8}\right) in = 20.1875\,in$$

$$\bar{y} = \frac{2 \times 1in^2 \times 4.0\,in + 1.5625\,in^2 \times 4.125\,in + 3 \times 1in^2 \times 18.16\,in + 3 \times 0.3906\,in^2 \times 12.3125\,in + 2 \times 0.3906\,in^2 \times 20.1875 \times in}{2 \times 1in^2 + 1.5625\,in^2 + 3 \times 1in^2 + 3 \times 0.3906\,in^2 + 2 \times 0.3906\,in^2} = 11.6404\,in$$

$$d = 24\,in - 11.6404\,in = 12.3596\,in$$

$$M_n^+ = 8.5155\,in^2 \times 33 \frac{kips}{in^2} \times \left[12.3596\,in - \left(\frac{2.3614\,in}{2}\right)\right] \times \frac{1\,ft}{12\,in} = 261.7833\,k - ft$$

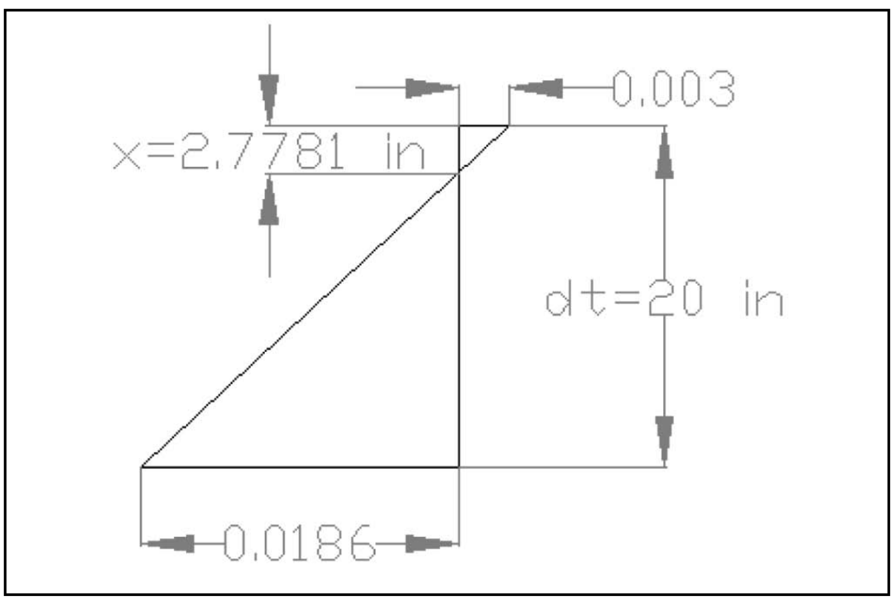


Figure C4. Strain diagram of the exterior beam at mid-span.

$$\varepsilon_t = \varepsilon_c \times \frac{d_t - x}{x}$$

where:

$$\varepsilon_c = Ultimate\ Strain\ of\ Concrete = 0.003$$
$$d_t = Effective\ Depth\ of\ the\ Extreme\ Tension\ Steel$$
$$d_t = h - \left(Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8\right) = 24in - \left[3.5in + \frac{1}{2} \times \left(\frac{8}{8}\right)in\right] = 20in$$
$$\varepsilon_t = 0.003 \times \frac{20in - 2.7781in}{2.7781in} = 0.01860\frac{in}{in} > 0.005\frac{in}{in}, therefore,$$
$$Tension - controlled\ and\ \phi = 0.9$$

Capacities for the load rating are

$$\phi \times M_n^+ = 0.9 \times 261.7833k - ft = 235.6050k - ft \quad \text{(LFR capacity)}$$

$$M_u^+ = 261.7833k - ft \quad \text{(LRFR capacity)}$$

Nominal negative moment for the exterior beam at the edge of a pier

Assume that the bottom layer of three No. 8 reinforcement steel bars carries compression, and the rest of the reinforcement steel carries all tension (Figure C5).

Check if section is composite:

$$A_s \le \frac{0.85 \times f'_c \times b_e \times t}{f_y}$$

$$A_s = A_{s1} + A_{s2} + A_{s3} = 2\#5 + 3\#5 + 3\#8$$

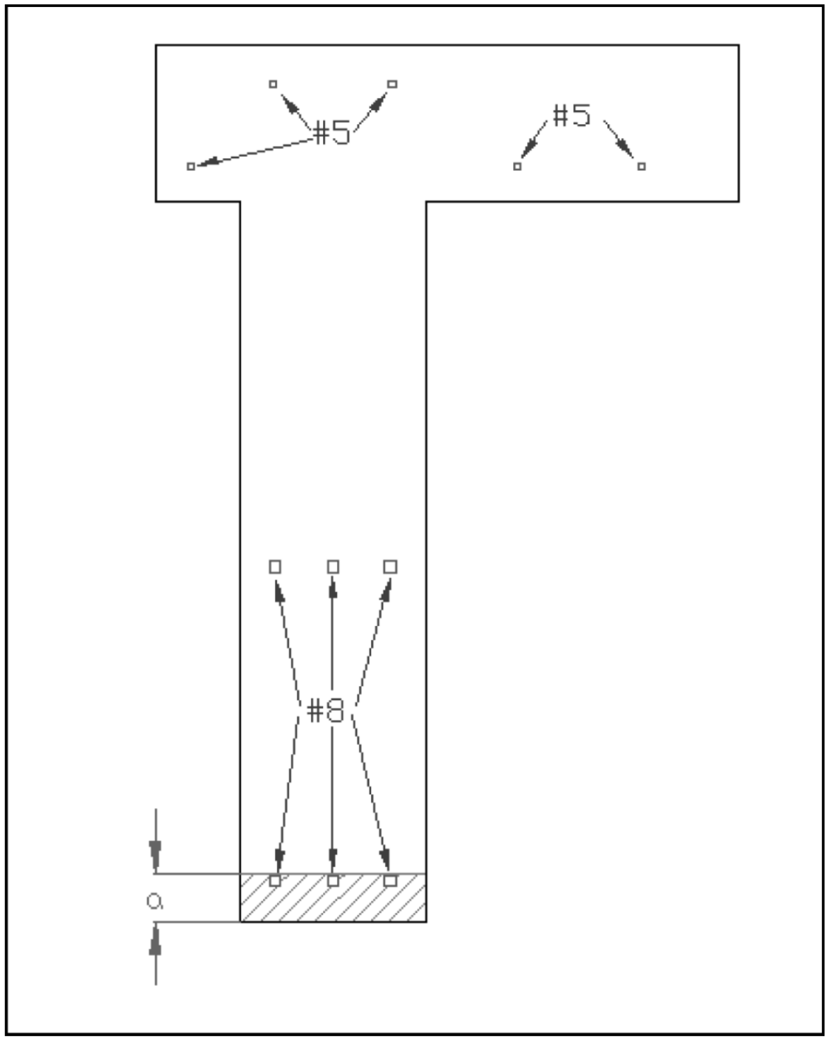


Figure C5. Cross section of the exterior beam at end-span.

where:

$$A_{s\#5} = \left(\frac{5}{8}\right) in \times \left(\frac{5}{8}\right) in = 0.3906\, in^2$$

$$A_{s\#8} = \left(\frac{8}{8}\right) in \times \left(\frac{8}{8}\right) in = 1.0\, in^2$$

$$A_{s1} = 2 \times 0.3906\, in^2 = 0.7812\, in^2$$

$$A_{s2} = 3 \times 0.3906\, in^2 = 1.1718\, in^2$$

$$A_{s3} = 3 \times 1 in^2 = 3 in^2$$

$$A_s = 0.7812\,in^2 + 1.1718\,in^2 + 3\,in^2 = 4.953\,in^2$$

$$\frac{0.85\times f'_c \times b_e \times t}{f_y} = \frac{0.85\times 2.5\dfrac{kips}{in^2}\times 18\,in \times 15\,in}{33\dfrac{kips}{in^2}} = 17.3864\,in^2$$

Since $A_s < \dfrac{0.85\times f'_c \times b_e \times t}{f_y}$, the section is composite; rectangular beam formulas are now valid.

$$T = A_s \times f_y = 4.953\,in^2 \times 33\frac{kips}{in^2} = 163.449\,kips$$

$$C = T \Rightarrow 0.85\times f'_c \times b_e \times a = T \Rightarrow a = \frac{T}{0.85\times f'_c \times b_e} = \frac{163.449\,kips}{0.85\times 2.5\dfrac{kips}{in^2}\times 18\,in} = 4.2732\,in > 3.5\,in$$

Therefore, the bottom layer of reinforcement steel (three No. 8) carries compression, while the rest of the reinforcement steel carries all tension.

Figure C6 shows that the assumption (i.e., the bottom layer of three No. 8 reinforcement steel carrying compression and the rest of the reinforcement steel carrying all tension) was correct.

$$x = \frac{a}{\beta_1} \Rightarrow \beta_1 = 0.85 \;\; if \;\; f'_c \le 4\,ksi \Rightarrow x = \frac{4.2732\;in}{0.85} = 5.0273\,in$$

$$M_n^- = A_s \times f_y \times \left[d - \left(\frac{a}{2}\right)\right] \Rightarrow d = h - \bar{y}$$

$$\bar{y} = \frac{2\times A_{s\#5}\times d_1 + 3\times A_{s\#5}\times d_2 + 3\times A_{s\#8}\times d_3}{2\times A_{s\#5} + 3\times A_{s\#5} + 3\times A_{s\#8}}$$

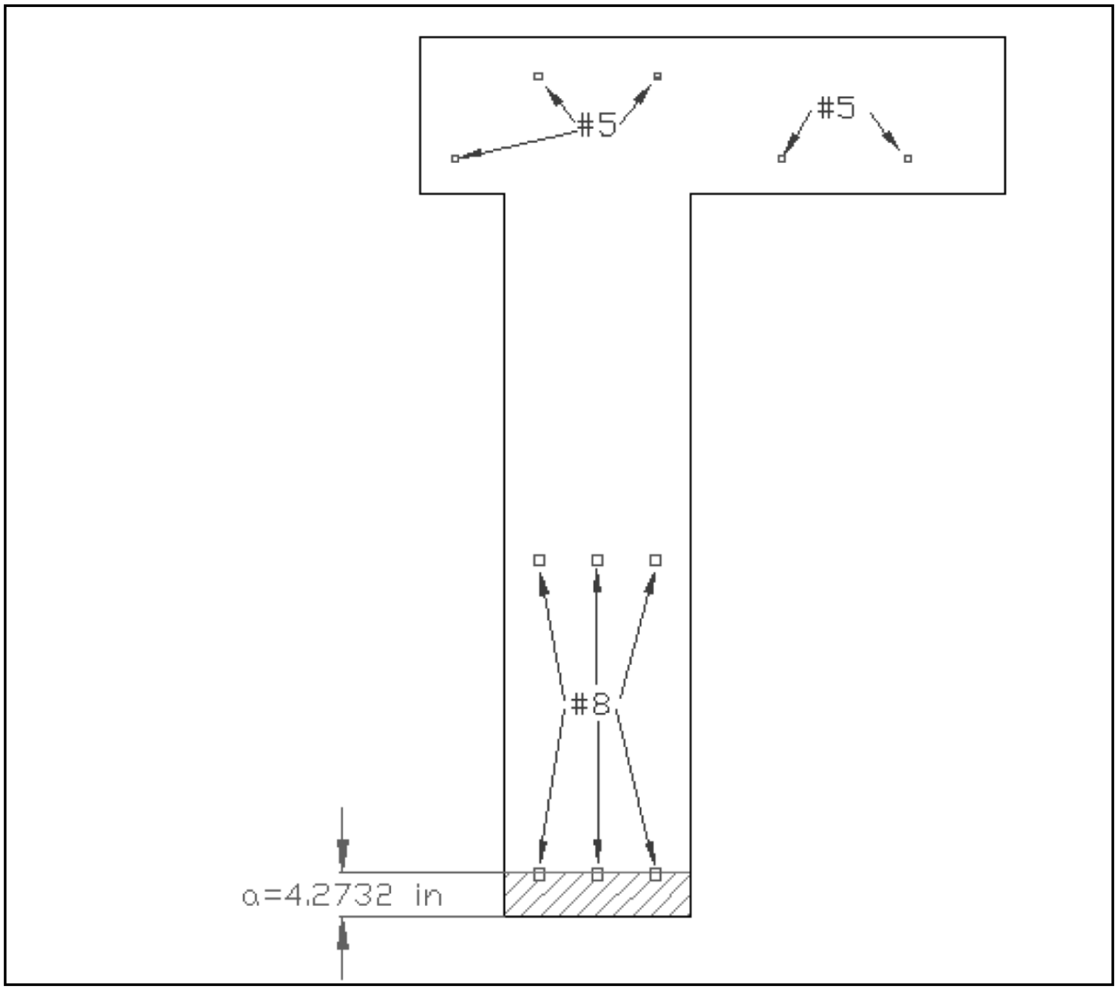


Figure C6. Cross section of the exterior beam at end-span with the steel rebars.

Figure C7 shows the distances that were needed to calculate the centroid of the reinforcement steel to calculate the negative flexural capacity.
Figure C8 presents a strain diagram of the exterior beam at end-span.

where:

$$d_1 = Cover\ of\ the\ top\ of\ the\ slab + \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 3.5\,in + \frac{1}{2} \times \left(\frac{5}{8}\right) in = 3.8125\,in$$

$$d_2 = t_s - Cover\ of\ the\ bottom\ of\ the\ slab - \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 15\,in - 3\,in - \frac{1}{2} \times \left(\frac{5}{8}\right) in = 11.6875\,in$$

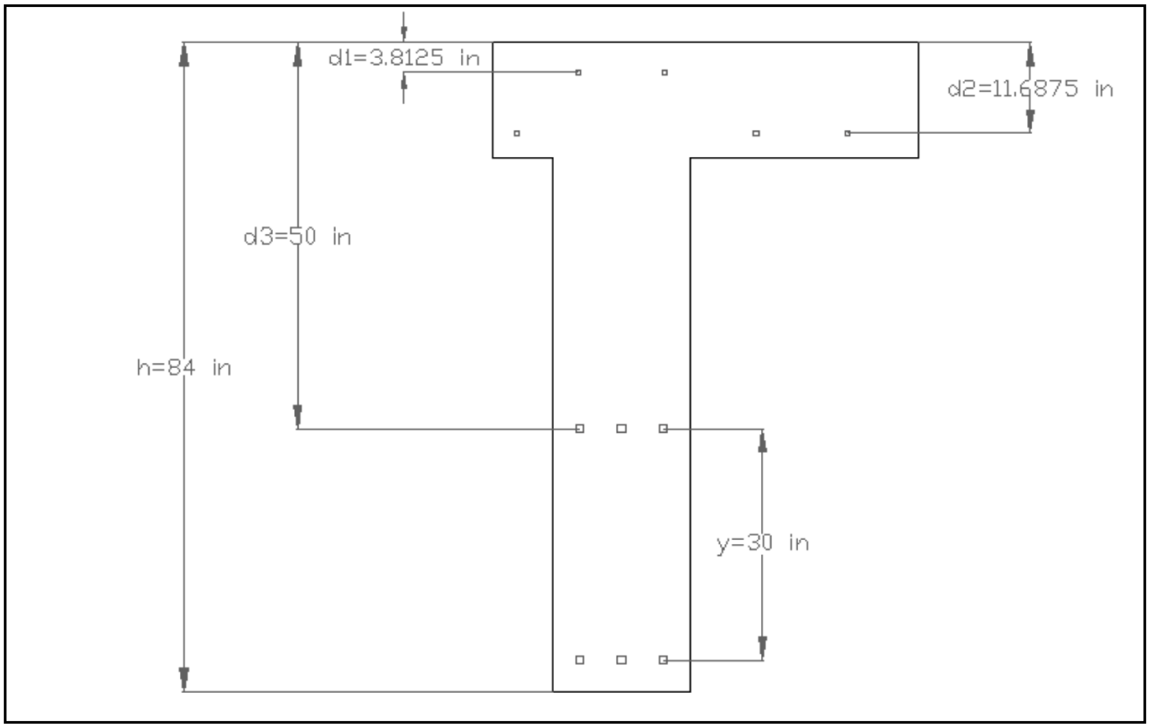


Figure C7. Cross section of the exterior beam at end-span, rebars locations.

$$d_3 = h - \left(Cover\ of\ the\ bottom\ of\ the\ slab + \frac{1}{2} \times height\ of\ a\ bar\ \#8 + y \right) =$$

$$d_3 = 84\,in - \left(3.5\,in + \frac{1}{2} \times \left(\frac{8}{8} \right) in + 30\,in \right) = 50\,in$$

$$\bar{y} = \frac{2 \times 0.3906\,in^2 \times 3.8125\,in + 3 \times 0.3906\,in^2 \times 11.6875\,in + 3 \times 1\,in^2 \times 50\,in}{2 \times 0.3906\,in^2 + 3 \times 0.3906\,in^2 + 3 \times 1\,in^2} = 33.6511\,in$$

$$d = 84\,in - 33.6511\,in = 50.3489\,in$$

$$M_n^- = 4.953\,in^2 \times 33 \frac{kips}{in^2} \times \left[50.3489\,in - \left(\frac{4.2732\,in}{2} \right) \right] \times \frac{1\,ft}{12\,in} = 656.6877\,k - ft$$

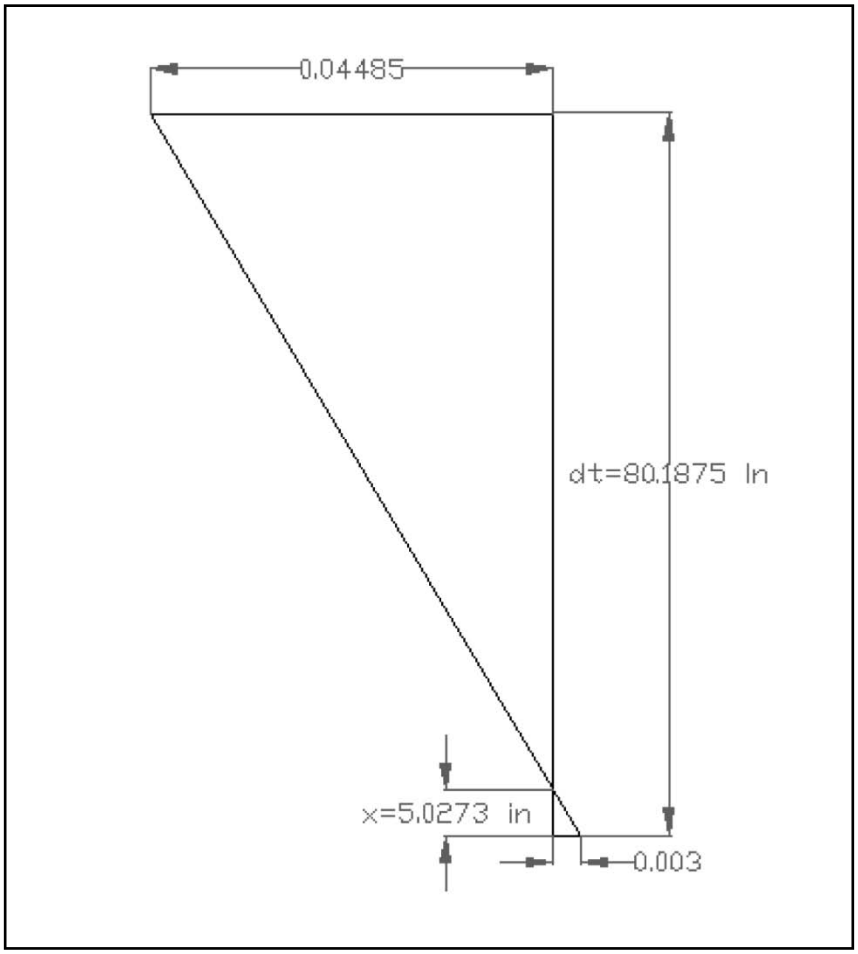


Figure C8. Strain diagram of the exterior beam at end-span.

$$\varepsilon_t = \varepsilon_c \times \frac{d_t - x}{x}$$

where:

$$\varepsilon_c = Ultimate\,Strain\,of\,\,Concrete = 0.003$$
$$d_t = Effective\,Depth\,of\,\,the\,Extreme\,Tension\,Steel =$$
$$h - \left(Cover\,of\,\,the\,bottom\,of\,\,the\,beam + \frac{1}{2} \times height\,of\,\,a\,bar\,\#5 \right) = 84\,in - \left[3.5\,in + \frac{1}{2} \times \left(\frac{5}{8} \right) in \right] = 80.1875\,in$$
$$\varepsilon_t = 0.003 \times \frac{80.1875in - 5.0273\,in}{5.0273\,in} = 0.04485\frac{in}{in} > 0.005\frac{in}{in},$$
$$Therefore, is\,tension - controlled\,\,and\,\,\phi = 0.9$$

Capacities for the load rating are

$\phi \times M_n^- = 0.9 \times 656.6877\,k - ft = 591.0189\,k - ft$ (LFR capacity)

$M_u^- = 656.6877\,k - ftt$ (LRFR capacity)

LFR shear capacity at the edge section of the exterior beam

$$\phi \times V_n = \phi \times (V_c + V_s)$$

$$V_c = 2 \times \sqrt{f'_c} \times b_w \times d$$

where:

$$d = h - \overline{y}$$

$$d = 50.34 in$$

$$V_c = 2 \times \sqrt{2500 \frac{lb}{in^2}} \times 18\,in \times 50.34\,in \times \frac{1\,kip}{1000\,lb} = 90.61\,kips$$

$$V_s = \frac{A_v \times f_y \times d}{s}$$

where:

$$A_v = 2 \times A_{s\#4}$$

$$A_{s\#4} = \left(\frac{4}{8}\right) in \times \left(\frac{4}{8}\right) in = 0.25\,in^2$$

$$A_v = 2 \times 0.25\,in^2 = 0.50\,in^2$$

$$V_s = \frac{0.50\,in^2 \times 33 \frac{kips}{in^2} \times 50.34\,in}{29\,in} = 28.64\,kips$$

Capacity for the LFR is

$$\phi \times V_n = 0.85 \times (90.61 kips + 28.64 kips) = 101.38 kips$$

LRFD shear capacity at the edge section of the exterior beam

Nominal shear resistance is given as

$$V_n = (V_c + V_s)$$

for which

$$V_c = 0.0316 \times \beta \times \sqrt{f'_c} \times b_v \times d_v \qquad \text{(LRFD 5-68)}$$

$$V_s = \frac{A_v \times f_y \times d_v \times \cot\theta}{s} \qquad \text{(LRFD 5-69)}$$

$$\beta = 2.0 \qquad \text{(LRFD 5.8.3.4)}$$

$$\theta = 45^\circ \qquad \text{(LRFD 5.8.3.4)}$$

$$V_c = 0.0316 \times 2 \times \sqrt{2.5 ksi} \times 18 in \times 50.3489 in = 90.56 kips$$

No. 4 shear stirrups at 29-in. spacing are provided along the beam.

$$V_s = \frac{0.50 in^2 \times 33 ksi \times 50.3489 in \times \cot(45)}{29 in} = 28.64 kips$$

LRFD design shear capacity is given by

$$V_n = (90.56 kips + 28.64 kips) = 119.20 kips$$

Interior beam

Nominal positive moment for the interior beam at the mid-span location

Assume that all reinforcement steel carries all tension.

Figure C9 shows the assumption of all the reinforcement steel carrying all tension.

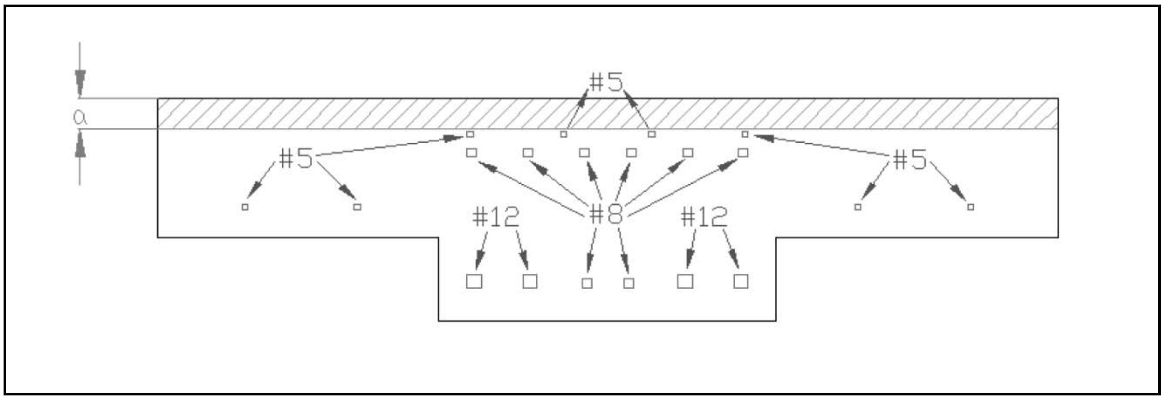


Figure C9. Cross section of the interior beam at mid-span with steel rebar size.

Check if section is composite:

$$A_s \leq \frac{0.85 \times f'_c \times b_e \times t}{f_y}$$

$$A_s = A_{s1} + A_{s2} + A_{s3} + A_{s4} = (4\#12 + 2\#8) + 6\#8 + 4\#5 + 4\#5$$

where:

$$A_{s\#5} = \left(\frac{5}{8}\right) in \times \left(\frac{5}{8}\right) in = 0.3906\, in^2$$

$$A_{s\#8} = \left(\frac{8}{8}\right) in \times \left(\frac{8}{8}\right) in = 1.0\, in^2$$

$$A_{s\#12} = \left(\frac{12}{8}\right) in \times \left(\frac{12}{8}\right) in = 2.25\, in^2$$

$$A_{s1} = 4 \times 2.25\, in^2 + 2 \times 1\, in^2 = 11\, in^2$$

$$A_{s2} = 6 \times 1 in^2 = 6\, in^2$$

$$A_{s3} = 4 \times 0.3906\, in^2 = 1.5624\, in^2$$

$$A_{s4} = 4 \times 0.3906\, in^2 = 1.5624\, in^2$$

$$A_s = 11in^2 + 6in^2 + 1.5624in^2 + 1.5624in^2 = 20.1248in^2$$

$$\frac{0.85 \times f'_c \times b_e \times t}{f_y} = \frac{0.85 \times 2.5\frac{kips}{in^2} \times 96\,in \times 15\,in}{33\frac{kips}{in^2}} = 92.7273\,in^2$$

Since $A_s < \dfrac{0.85 \times f'_c \times b_e \times t}{f_y}$, the section is composite; rectangular beam formulas are now valid.

$$T = A_s \times f_y = 20.1248in^2 \times 33\frac{kips}{in^2} = 664.1184\,kips$$

$$C = T \Rightarrow 0.85 \times f'_c \times b_e \times a = T \Rightarrow a = \frac{T}{0.85 \times f'_c \times b_e} = \frac{664.1184\,kips}{0.85 \times 2.5\frac{kips}{in^2} \times 96\,in} = 3.2555\,in < 3.5\,in$$

Therefore, all the reinforcement steel carries all tension.

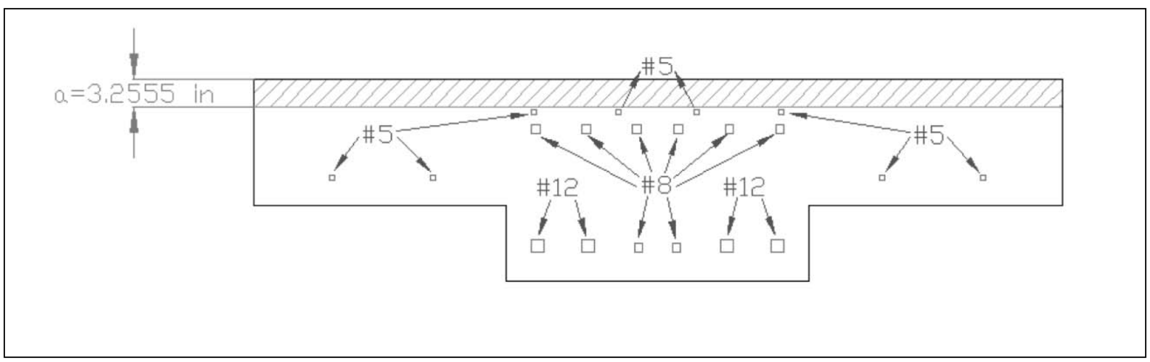


Figure C10. Cross section of the interior beam at mid-span showing the compression zone.

Figure C10 shows that the assumption of all the reinforcement steel carrying all tension was correct.

$$x = \frac{a}{\beta_1} \Rightarrow \beta_1 = 0.85 \;\; if \;\; f'_c \le 4\,ksi \Rightarrow x = \frac{3.2555\;in}{0.85} = 3.83\,in$$

$$M_n^+ = A_s \times f_y \times \left[d - \left(\frac{a}{2}\right)\right] \Rightarrow d = h - \overline{y}$$

$$\bar{y} = \frac{4 \times A_{s\#12} \times d_1 + 2 \times A_{s\#8} \times d_2 + 6 \times A_{s\#8} \times d_3 + 4 \times A_{s\#5} \times d_4 + 4 \times A_{s\#5} \times d_5}{4 \times A_{s\#12} + 2 \times A_{s\#8} + 6 \times A_{s\#8} + 4 \times A_{s\#5} + 4 \times A_{s\#5}}$$

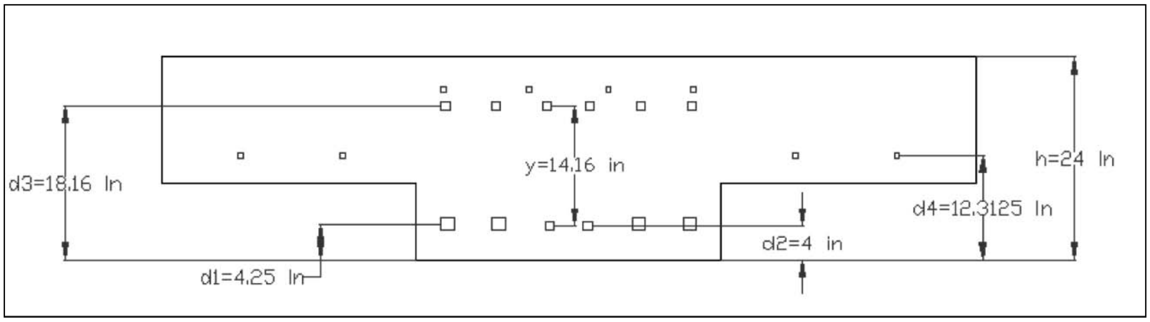


Figure C11. Cross section of the interior beam at mid-span steel rebar locations.

Figure C11 shows the distances that were needed to calculate the centroid of the reinforcement steel for use in calculating the positive flexural capacity. Figure C12 presents a strain diagram of the interior beam at mid-span.

where:

$$d_1 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#12 = 3.5\,in + \frac{1}{2} \times \left(\frac{12}{8}\right) in = 4.2500\,in$$

$$d_2 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8 = 3.5\,in + \frac{1}{2} \times \left(\frac{8}{8}\right) in = 4.0000\,in$$

$$d_3 = Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\ \#8 + y$$

$$d_3 = 3.5\,in + \frac{1}{2} \times \left(\frac{8}{8}\right) in + 14.16\,in = 18.1600\,in$$

$$d_4 = h - (t_s + Cover\ of\ the\ bottom\ of\ the\ slab) + \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 24\,in - (15\,in - 3\,in) +$$
$$\frac{1}{2} \times \left(\frac{5}{8}\right) in = 12.3125\,in$$

$$d_5 = h - Cover\ of\ the\ top\ of\ the\ slab - \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 24\,in - 3.5\,in - \frac{1}{2} \times \left(\frac{5}{8}\right) in = 20.1875\,in$$

$$\bar{y} = \frac{4 \times 2.25\,in^2 \times 4.25\,in + 2 \times 1\,in^2 \times 4.00\,in + 6 \times 1\,in^2 \times 18.16\,in + 4 \times 0.3906\,in^2 \times 12.3125\,in + 4 \times 0.3906\,in^2 \times 20.1875\,in}{4 \times 2.25\,in^2 + 2 \times 1\,in^2 + 6 \times 1\,in^2 + 4 \times 0.3906\,in^2 + 4 \times 0.3906\,in^2} = 10.2355\,in$$

$$d = 24\,in - 10.2355\,in = 13.7645\,in$$

$$M_n^+ = 20.1248\,in^2 \times 33\frac{kips}{in^2} \times \left[13.7645\,in - \left(\frac{3.2555\,in}{2}\right)\right] \times \frac{1\,ft}{12\,in} = 671.6866\,k - ft$$

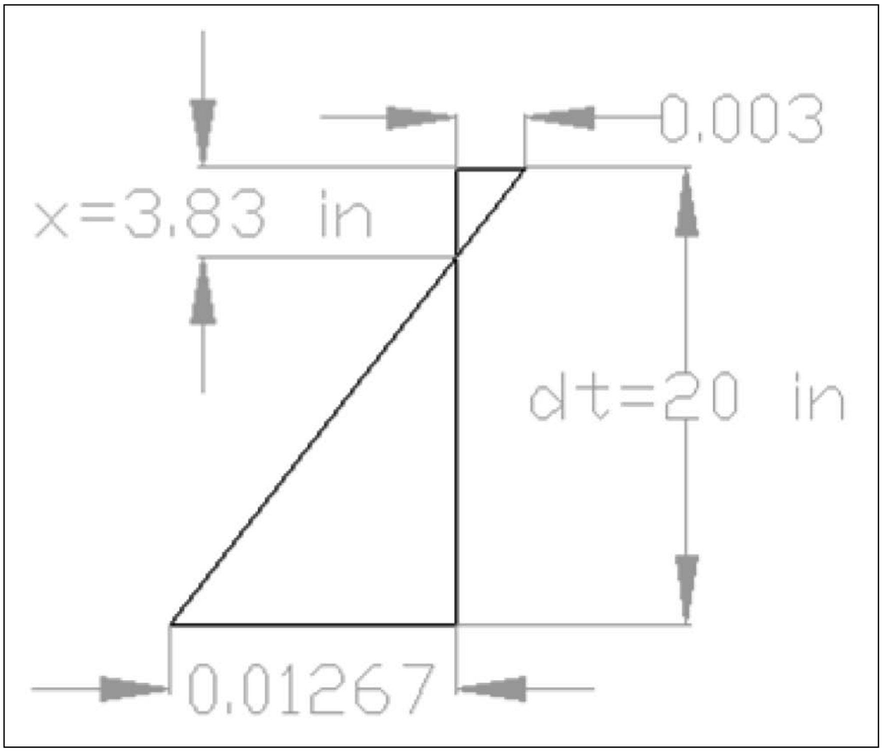


Figure C12. Strain diagram of the interior beam at mid-span.

$$\varepsilon_t = \varepsilon_c \times \frac{d_t - x}{x}$$

and where:

$$\varepsilon_c = Ultimate\ Strain\ of\ Concrete = 0.003$$
$$d_t = Effective\ Depth\ of\ the\ Extreme\ Tension\ Steel =$$
$$h - \left(Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar\right) = 24\,in - \left[3.5\,in + \frac{1}{2} \times \left(\frac{8}{8}\right) in\right] = 20\,in$$
$$\varepsilon_t = 0.003 \times \frac{20in - 3.83\,in}{3.83\,in} = 0.01267\frac{in}{in} > 0.005\frac{in}{in},$$
$$Therefore, is\ tension - controlled\ and\ \phi = 0.9$$

Capacities for the load rating are

$$\phi \times M_n^+ = 0.9 \times 671.6866\,k - ft = 604.5179\,k - ft \quad \text{(LFR capacity)}$$

$M_u^+ = 671.6866\, k - ft$ (LRFR capacity)

Nominal negative moment for the interior beam at the edge location

Assume that the bottom layer of four No. 12 and two No. 8 reinforcement steel bars carries compression and the rest of the reinforcement steel carries all tension (Figure C13).

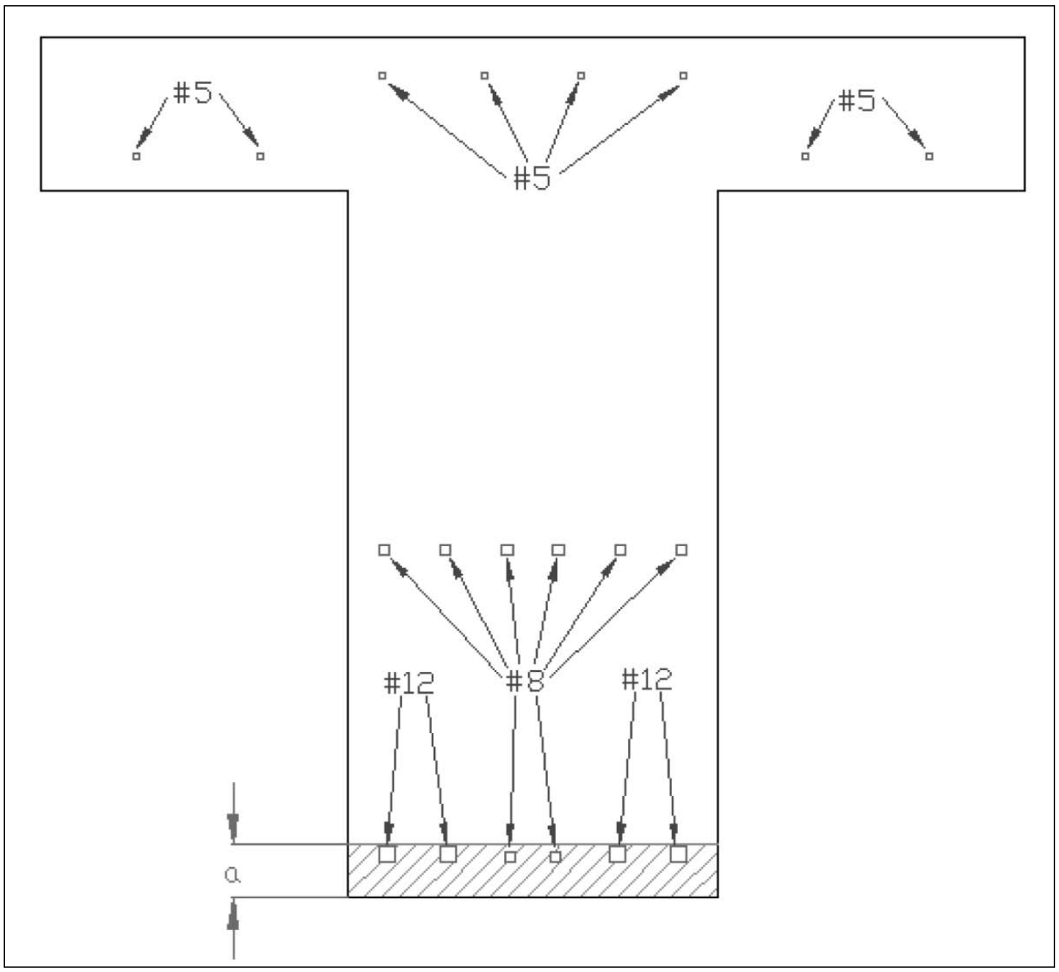


Figure C13. Cross section of the interior beam at end-span with the steel rebar size.

Check if section is composite:

$$A_s \le \frac{0.85 \times f'_c \times b_e \times t}{f_y}$$

$$A_s = A_{s1} + A_{s2} + A_{s3} = 4\#5 + 4\#5 + 6\#8$$

where:

$$A_{s\#5} = \left(\frac{5}{8}\right) in \times \left(\frac{5}{8}\right) in = 0.3906\, in^2$$

$$A_{s\#8} = \left(\frac{8}{8}\right) in \times \left(\frac{8}{8}\right) in = 1.0\, in^2$$

$$A_{s1} = 4 \times 0.3906\, in^2 = 1.5624\, in^2$$

$$A_{s2} = 4 \times 0.3906\, in^2 = 1.5624\, in^2$$

$$A_{s3} = 6 \times 1 in^2 = 6\, in^2$$

$$A_s = 1.5624\, in^2 + 1.5624\, in^2 + 6\, in^2 9.1248\, in^2$$

$$\frac{0.85 \times f'_c \times b_e \times t}{f_y} = \frac{0.85 \times 2.5 \frac{kips}{in^2} \times 36\, in \times 15\, in}{33 \frac{kips}{in^2}} = 34.7727\, in^2$$

Since $A_s < \dfrac{0.85 \times f'_c \times b_e \times t}{f_y}$, the section is composite; rectangular beam formulas are now valid.

$$T = A_s \times f_y = 9.1248 in^2 \times 33 \frac{kips}{in^2} = 301.1184\, kips$$

$$C = T \Rightarrow 0.85 \times f'_c \times b_e \times a = T \Rightarrow a = \frac{T}{0.85 \times f'_c \times b_e} = \frac{301.1184\, kips}{0.85 \times 2.5 \frac{kips}{in^2} \times 36\, in} = 3.9362\, in > 3.5\, in$$

Therefore, the bottom layer of reinforcement steel, four No. 12 and two No. 8, carries compression, while the rest of the reinforcement steel carries all tension.

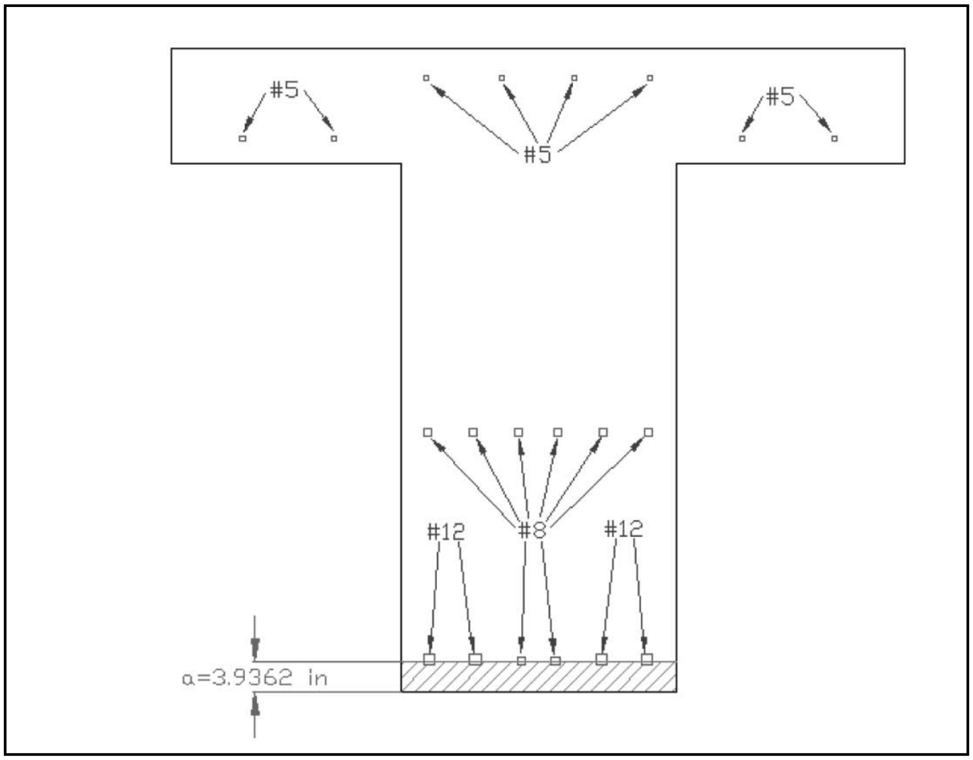


Figure C14. Cross section of the interior beam at end-span.

Figure C14 shows that the assumption (i.e., the bottom layer of four No. 12 and two No. 8 reinforcement steel carrying compression and the rest of the reinforcement steel carrying all tension) was correct.

$$x = \frac{a}{\beta_1} \Rightarrow \beta_1 = 0.85 \ \ if \ \ f'_c \leq 4ksi \Rightarrow x = \frac{3.9362 \ in}{0.85} = 4.6308 in$$

$$M_n^- = A_s \times f_y \times \left[d - \left(\frac{a}{2} \right) \right] \Rightarrow d = h - \overline{y}$$

$$\overline{y} = \frac{4 \times A_{s\#5} \times d_1 + 4 \times A_{s\#5} \times d_2 + 6 \times A_{s\#8} \times d_3}{4 \times A_{s\#5} + 4 \times A_{s\#5} + 6 \times A_{s\#8}}$$

Figure C15 shows the distances that were needed to calculate the centroid of the reinforcement steel for use in calculating the negative flexural capacity. Figure C16 presents a strain diagram of the interior beam at mid-span.

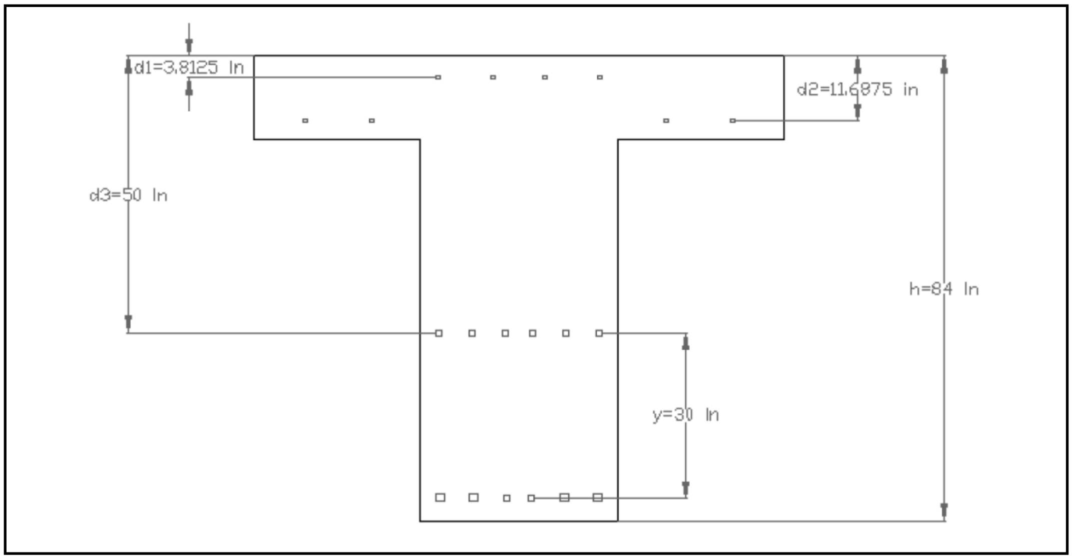


Figure C15. Cross section of the interior beam at end-span, steel rebars location.

where:

$$d_1 = Cover\ of\ the\ top\ of\ the\ slab + \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 3.5\,in + \frac{1}{2} \times \left(\frac{5}{8}\right) in = 3.8125\,in$$

$$d_2 = t_s - Cover\ of\ the\ bottom\ of\ the\ slab - \frac{1}{2} \times height\ of\ a\ bar\ \#5 = 15\,in - 3\,in - \frac{1}{2} \times \left(\frac{5}{8}\right) in = 11.6875\,in$$

$$d_3 = h - \left(Cover\ of\ the\ bottom\ of\ the\ slab + \frac{1}{2} \times height\ of\ a\ bar\ \#8 + y\right)$$

$$d_3 = 84\,in - \left(3.5\,in + \frac{1}{2} \times \left(\frac{8}{8}\right) in + 30\,in\right) = 50\,in$$

$$\bar{y} = \frac{4 \times 0.3906\,in^2 \times 3.8125\,in + 4 \times 0.3906\,in^2 \times 11.6875\,in + 6 \times 1\,in^2 \times 50\,in}{4 \times 0.3906\,in^2 + 4 \times 0.3906\,in^2 + 6 \times 1\,in^2} = 35.5314\,in$$

$$d = 84\,in - 35.5314\,in = 48.4686\,in$$

$$M_n^- = 9.1248\,in^2 \times 33 \frac{kips}{in^2} \times \left[48.4686\,in - \left(\frac{3.9362\,in}{2}\right)\right] \times \frac{1\,ft}{12\,in} = 1166.8463\,k - ft$$

$$\varepsilon_t = \varepsilon_c \times \frac{d_t - x}{x}$$

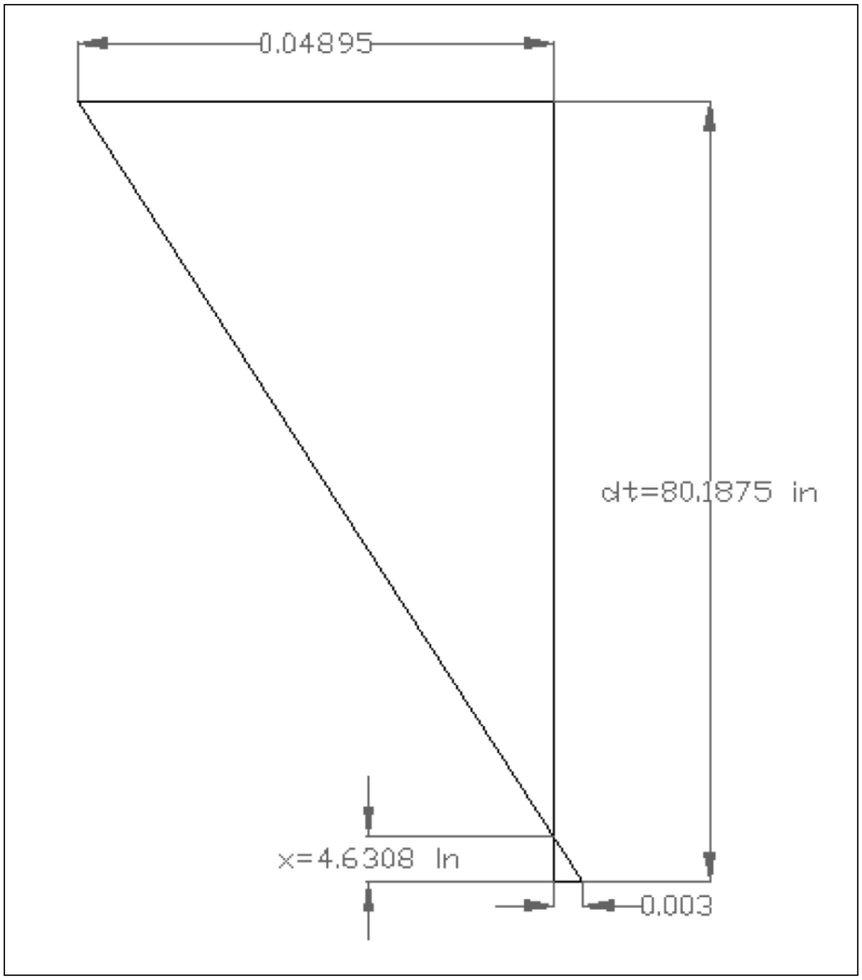


Figure C16. Strain diagram of the interior beam at mid-span.

and where:

$$\varepsilon_c = Ultimate\ Strain\ of\ Concrete = 0.003$$

$$d_t = Effective\ Depth\ of\ the\ Extreme\ Tension\ Steel =$$

$$h - \left(Cover\ of\ the\ bottom\ of\ the\ beam + \frac{1}{2} \times height\ of\ a\ bar \right) = 84\,in - \left[3.5\,in + \frac{1}{2} \times \left(\frac{5}{8} \right) in \right] = 80.1875\,in$$

$$\varepsilon_t = 0.003 \times \frac{80.1875\,in - 4.6308\,in}{4.6308\,in} = 0.04895\frac{in}{in} > 0.005\frac{in}{in},$$

$$Therefore, is\ tension - controlled\ and\ \phi = 0.9$$

Capacities for the load rating are

$$\phi \times M_n^- = 0.9 \times 1166.8463\, k - ft = 1050.1617\, k - ft \quad \text{(LFR capacity)}$$

$$M_u^- = 1166.8463\, k - ft \quad \text{(LRFR capacity)}$$

LFR nominal shear capacity of the interior beam at the edge

$$\phi \times V_n = \phi \times (V_c + V_s)$$

$$V_c = 2 \times \sqrt{f'_c} \times b_w \times d$$

where:

$$d = h - \overline{y}$$

$$d = 48.46in$$

$$V_c = 2 \times \sqrt{2500 \frac{lb}{in^2}} \times 36\, in \times 48.469 in \times \frac{1kip}{1000\, lb} = 174.456\, kips$$

$$V_s = \frac{A_v \times f_y \times d}{s}$$

where:

$$A_v = 2 \times A_{s\#4}$$

$$A_{s\#4} = \left(\frac{4}{8}\right) in \times \left(\frac{4}{8}\right) in = 0.25\, in^2$$

$$A_v = 2 \times 0.25\, in^2 = 0.50\, in^2$$

$$V_s = \frac{0.50\, in^2 \times 33 \frac{kips}{in^2} \times 48.469 in}{29\, in} = 27.57\, kips$$

LFR design shear capacity is given by

$$\phi \times V_n = 0.85 \times (174.456\,kips + 27.57\,kips) = 171.72\,kips$$

LRFD shear capacity at the edge section of the interior beam

Nominal shear resistance is given as

$$V_n = (V_c + V_s)$$

For which

$$V_c = 0.0316 \times \beta \times \sqrt{f'_c} \times b_v \times d_v \qquad \text{(LRFD 5-68)}$$

$$V_s = \frac{A_v \times f_y \times d_v \times \cot\theta}{s} \qquad \text{(LRFD 5-69)}$$

$$\beta = 2.0 \qquad \text{(LRFD 5.8.3.4)}$$

$$\theta = 45^{\circ} \qquad \text{(LRFD 5.8.3.4)}$$

$$V_c = 0.0316 \times 2 \times \sqrt{2.5\,ksi} \times 36in \times 48.469in = 174.36kips$$

No. 4 shear stirrups at 29-in. spacing are provided along the beam.

$$V_s = \frac{0.50in^2 \times 33ksi \times 48.469in \times \cot(45)}{29in} = 27.57kips$$

LRFD design shear capacity is given by

$$V_n = (174.36kips + 27.57kips) = 201.90kips$$

Appendix D: Rating Factor Example

Load factor (LFR) rating equation

$$RF = \frac{C - A_1 \times D}{A_2 \times L}$$

where:

$$C = Capacity\ of\ the\ beam$$

$$A_1 = Dead\ Load\ Factor \begin{pmatrix} 1.3\ for\ Operating\ and\ Inventory \\ 1\ for\ Allowable\ Stress \end{pmatrix}$$

$$D = Dead\ Load$$

$$A_2 = LiveLoadFactor \begin{pmatrix} 1.3\ for\ Operating; \\ 2.17\ for\ Inventory \end{pmatrix}$$

$$L = Live\ Load$$

LFR method: exterior beam

Load rating for the exterior beam at 1/3 of the mid-span length using the HS20 design vehicle

Negative moment:

From Table 7-1,

$$C = \phi \times M_n^- = 359.8497\, k - ft$$

From Table 5-1,

$$D = M_{DL}^- = 164.88\, k - ft$$

From Table 6-1

$$L = M_{LL}^- = 143.5\, k - ft$$

For the Operating Level rating:

$$RF = \frac{359.8497\,k-ft - (1.3 \times 164.88\,k-ft)}{1.3 \times 143.5\,k-ft} = 0.78$$

For the Inventory Level rating:

$$RF = \frac{359.8497\,k-ft - (1.3 \times 164.88\,k-ft)}{2.17 \times 143.5\,k-ft} = 0.47$$

Shear force:

From Table 7-1,

$$C = \phi \times V_n = 63.408\,k$$

From Table 5-1,

$$D = V_{DL} = 21.42\,k$$

From Table 6-1,

$$L = V_{LL} = 21.6252\,k$$

For the Operating Level rating:

$$RF = \frac{63.408\,k - (1.3 \times 21.42\,k)}{1.3 \times 21.6252\,k} = 1.27$$

For the Inventory Level rating:

$$RF = \frac{63.408\,k - (1.3 \times 21.42\,k)}{2.17 \times 21.6252\,k} = 0.76$$

LFR method: exterior beam

Load rating for the exterior beam at 2/3 of the mid-span length using the HS20 design vehicle:

Positive moment:

From Table 7-1,

$$C = \phi \times M_n^+ = 285.3778k - ft$$

From Table 5-1,

$$D = M_{DL}^+ = 40.11k - ft$$

From Table 6-1,

$$L = M_{LL}^- = 63.2901k - ft$$

For the Operating Level rating:

$$RF = \frac{285.3778k - ft - (1.3 \times 40.11k - ft)}{1.3 \times 63.2901k - ft} = 2.83$$

For the Inventory Level rating:

$$RF = \frac{285.3778k - ft - (1.3 \times 40.11k - ft)}{2.17 \times 63.2901k - ft} = 1.69$$

Shear force:

From Table 7-1,

$$C = \phi \times V_n = 31.412k$$

From Table 5-1,

$$D = V_{DL} = 11.89k$$

From Table 6-1

$$L = V_{LL} = 16.8205k$$

For the Operating Level rating:

$$RF = \frac{31.412k - (1.3 \times 11.89k)}{1.3 \times 16.8205k} = 0.73$$

For the Inventory Level rating:

$$RF = \frac{31.412\,k - (1.3 \times 11.89\,k)}{2.17 \times 16.8205\,k} = 0.44$$

LFR method: interior beam

Load rating for the interior beam at the edge using the HS20 design vehicle:

Negative moment:

From Table 7-3,

$$C = \phi \times M_n^- = 1050.1617\,k - ft$$

From Table 5-2,

$$D = M_{DL}^- = 618.49\,k - ft$$

From Table 6-2,

$$L = M_{LL}^- = 443.12k - ft$$

For the Operating Level rating:

$$RF = \frac{1050.1617\,k - ft - (1.3 \times 618.49\,k - ft)}{1.3 \times 443.12\,k - ft} = 0.43$$

For the Inventory Level rating:

$$RF = \frac{1050.1617\,k - ft - (1.3 \times 618.49\,k - ft)}{2.17 \times 443.12\,k - ft} = 0.26$$

Shear:

From Table 7-3,

$$C = \phi \times V_n = 171.73\,k$$

From Table 5-2,

$$D = V_{DL} = 59.26k$$

From Table 6-2,

$$L = V_{LL} = 44.5836k$$

For the Operating Level rating:

$$RF = \frac{171.73k - (1.3 \times 59.26k)}{1.3 \times 44.5836k} = 1.63$$

For the Inventory Level rating:

$$RF = \frac{171.73k - (1.3 \times 59.26k)}{2.17 \times 44.5836k} = 0.98$$

LFR method: interior beam

Load rating for the interior beam at mid-span using the HS20 design vehicle.

Positive moment:

From Table 7-3,

$$C = \phi \times M_n^+ = 604.5179k - ft$$

From Table 5-2,

$$D = M_{DL}^+ = 114.34k - ft$$

From Table 6-2,

$$L = M_{LL}^+ = 156.507k - ft$$

For the Operating Level rating:

$$RF = \frac{604.5179\,k-ft-(1.3\times 114.34\,k-ft)}{1.3\times 156.507\,k-ft} = 2.24$$

For the Inventory Level rating:

$$RF = \frac{604.5179\,k-ft-(1.3\times 114.34\,k-ft)}{2.17\times 156.507\,k-ft} = 1.34$$

Load rating for the interior beam at 1/3 of the mid-span length using the HS20 design vehicle:

LFR method: interior beam

Negative moment:

From Table 7-3,

$$C = \phi \times M_n^- = 653.5594\,k-ft$$

From Table 5-2,

$$D = M_{DL}^- = 288.44\,k-ft$$

From Table 6-2,

$$L = M_{LL}^- = 243.33\,k-ft$$

For the Operating Level rating:

$$RF = \frac{653.5594\,k-ft-(1.3\times 288.44\,k-ft)}{1.3\times 243.33\,k-ft} = 0.88$$

For the Inventory Level rating:

$$RF = \frac{653.5594\,k-ft-(1.3\times 288.44\,k-ft)}{2.17\times 243.33\,k-ft} = 0.53$$

Shear:

From Table 7-3,

$C = \phi \times V_n = 109.523k$

From Table 5-2,

$D = V_{DL} = 37.15k$

From Table 6-2,

$L = V_{LL} = 36.6687k$

For the Operating Level rating:

$$RF = \frac{109.523k - (1.3 \times 37.15k)}{1.3 \times 36.6687k} = 1.28$$

For the Inventory Level rating:

$$RF = \frac{109.523k - (1.3 \times 37.15k)}{2.17 \times 36.6687k} = 0.77$$

LFR method: interior beam

Load rating for the interior beam at 2/3 of the mid-span length using the HS20 design vehicle.

Positive moment:

From Table 7-3,

$C = \phi \times M_n^- = 826.6705k - ft$

From Table 5-2,

$D = M_{DL}^- = 66.81k - ft$

From Table 6-2,

$L = M_{LL}^- = 107.318k - ft$

For the Operating Level rating:

$$RF = \frac{826.6705k - ft - (1.3 \times 66.81k - ft)}{1.3 \times 107.318k - ft} = 5.30$$

For the Inventory Level rating:

$$RF = \frac{826.6705k - ft - (1.3 \times 66.81k - ft)}{2.17 \times 107.318k - ft} = 3.17$$

Shear:

From Table 7-3,

$$C = \phi \times V_n = 64.564k$$

From Table 5-2,

$$D = V_{DL} = 20.22k$$

From Table 6-2,

$$L = V_{LL} = 28.5218k$$

For the Operating Level rating:

$$RF = \frac{64.564k - (1.3 \times 20.22k)}{1.3 \times 28.5218k} = 1.03$$

For the Inventory Level rating:

$$RF = \frac{64.564k - (1.3 \times 20.22k)}{2.17 \times 28.5218k} = 0.62$$

Load and resistance factor rating (LRFR) equation

LRFD:

$$RF = \frac{C - \gamma_D \times D}{\gamma_L \times L}$$

where:

$$C = Capacity\ of\ the\ beam$$
$$\gamma_D = Dead\ Load\ Factor(1.25\ for\ Operating\ and\ Inventory\ on\ both\ Legal\ Loads\ and\ HL-93)$$
$$D = Dead\ Load$$
$$\gamma_L = Live Load Factor \begin{pmatrix} 1.35\ for\ Operating\ on\ HL-93 \\ 1.75\ for\ Inventory\ on\ HL-93 \\ 1.80\ on\ Legal\ Loads \end{pmatrix}$$
$$L = Live\ Load$$

LRFR method: exterior beam

Load rating for the exterior beam at the edge using the HL93 design vehicle:

Negative moment:

From Table 7-2,

$$C = M_n^- = 656.689\,k-ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n^- = 0.85 \times 1.0 \times 0.9 \times 656.689\,k-ft = 502.37k-ft$$

From Table 5-1,

$$D = M_{DL}^- = 354.28\,k-ft$$

From Table 6-21,

$$L = M_{LL}^- = 399.22\,k-ft$$

For the Operating Level rating:

$$RF = \frac{502.37\,k-ft-(1.25 \times 354.28\,k-ft)}{1.35 \times 399.22\,k-ft} = 0.11$$

For the Inventory Level rating:

$$RF = \frac{502.37k - ft - (1.25 \times 354.28\,k - ft)}{1.75 \times 399.22\,k - ft} = 0.09$$

Shear:

From Table 7-2,

$$C = V_n = 119.209\,k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 119.209k = 91.20\,k$$

From Table 5-1,

$$D = V_{DL} = 33.68\,k$$

From Table 6-21,

$$L = V_{LL} = 38.07\,k$$

For the Operating Level rating:

$$RF = \frac{91.20\,k - (1.25 \times 33.68\,k)}{1.35 \times 38.07\,k} = 0.96$$

For the Inventory Level rating:

$$RF = \frac{91.20\,k - (1.25 \times 33.68\,k)}{1.75 \times 38.07\,k} = 0.74$$

Since the HL-93 load rating is less than 1 for the negative moment, it is necessary to perform the analysis using the legal load vehicles.

Using the Type 3 (Legal Load): exterior beam

Negative moment:

From Table 7-2,

$$C = M_n^- = 656.689\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n^- = 0.85 \times 1.0 \times 0.9 \times 656.689\,k - ft = 502.37k - ft$$

From Table 5-1,

$$D = M_{DL}^- = 354.28k - ft$$

From Table 6-7,

$$L = M_{LL}^- = 201.56\,k - ft$$

$$RF = \frac{502.37k - ft - (1.25 \times 354.28\,k - ft)}{1.80 \times 201.56\,k - ft} = 0.16$$

Shear:

From Table 7-2,

$$C = V_n = 119.209\,k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 119.209k = 91.20\,k$$

From Table 5-1,

$$D = V_{DL} = 33.68\,k$$

From Table 6-7,

$$L = V_{LL} = 19.8636\,k$$

$$RF = \frac{91.20k - (1.25 \times 33.68\,k)}{1.80 \times 19.8636\,k} = 1.37$$

LRFR method: exterior beam

Load rating for the exterior beam at the mid-span using the HL93 design vehicle:

Positive moment:

From Table 7-2,

$$C = M_n^+ = 261.777\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 261.777k - ft = 200.26\,k - ft$$

From Table 5-1,

$$D = M_{DL}^+ = 69.10\,k - ft$$

From Table 6-21,

$$L = M_{LL}^- = 127.66\,k - ft$$

For the Operating Level rating:

$$RF = \frac{200.26\,k - ft - (1.25 \times 69.10\,k - ft)}{1.35 \times 127.66\,k - ft} = 0.66$$

For the Inventory Level rating:

$$RF = \frac{200.26\,k - ft - (1.25 \times 69.10\,k - ft)}{1.75 \times 127.66\,k - ft} = 0.51$$

Since the HL93 load rating is less than 1 for the positive moment, it is necessary to perform the analysis using the legal load vehicles.

Using the Type 3 (Legal Load)

Positive moment:

From Table 7-2,

$$C = M_n^+ = 261.777\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 261.777 k - ft = 200.26\,k - ft$$

From Table 5-1,

$$D = M_{DL}^+ = 69.10\,k - ft$$

From Table 6-7,

$$L = M_{LL}^- = 73.1269\,k - ft$$

$$RF = \frac{200.26\,k - ft - (1.25 \times 69.10\,k - ft)}{1.80 \times 73.1269\,k - ft} = 0.87$$

LRFR method: exterior beam

Load rating for the exterior beam at 1/3 of the mid-span length using the HL93 design vehicle:

Negative moment:

From Table 7-2,

$$C = M_n^- = 399.83\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 399.83 k - ft = 305.23\,k - ft$$

From Table 5-1,

$$D = M^{-}_{DL} = 164.88\,k - ft$$

From Table 6-21,

$$L = M^{-}_{LL} = 212.14\,k - ft$$

For the Operating Level rating:

$$RF = \frac{305.23\,k - ft - (1.25 \times 164.88\,k - ft)}{1.35 \times 212.14\,k - ft} = 0.35$$

For the Inventory Level rating:

$$RF = \frac{305.23k - ft - (1.25 \times 164.88\,k - ft)}{1.75 \times 212.14\,k - ft} = 0.27$$

Shear:

From Table 7-2,

$$C = V_n = 74.557\,k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 74.557k = 57.04\,k$$

From Table 5-1,

$$D = V_{DL} = 21.42\,k$$

From Table 6-21,

$$L = V_{LL} = 30.02\,k$$

For the Operating Level rating:

$$RF = \frac{57.04k - (1.25 \times 21.42\,k)}{1.35 \times 30.02k} = 0.75$$

For the Inventory Level rating:

$$RF = \frac{57.04\,k - (1.25 \times 21.42\,k)}{1.75 \times 30.02k} = 0.58$$

Since the HL-93 load rating is less than 1 for the negative moment, it is necessary to perform the analysis using the legal load vehicles.

Using the Type 3 (Legal Load)

Negative moment:

From Table 7-2,

$$C = M_n^- = 399.83k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 399.83k - ft = 305.23k - ft$$

From Table 5-1,

$$D = M_{DL}^- = 164.88k - ft$$

From Table 6-7,

$$L = M_{LL}^- = 115.52k - ft$$

$$RF = \frac{305.23k - ft - (1.25 \times 164.88k - ft)}{1.80 \times 115.52\,k - ft} = 0.48$$

Shear:

From Table 7-2,

$$C = V_n = 74.557\,k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 74.557 = 57.04\,k$$

From Table 5-1,

$$D = V_{DL} = 21.42\,k$$

From Table 6-7,

$$L = V_{LL} = 16.7205\,k$$

$$RF = \frac{57.05k - (1.25 \times 21.42\,k)}{1.80 \times 16.7205\,k} = 1.00$$

LRFR method: exterior beam

Load rating for the exterior beam at 2/3 of the mid-span length using the HL93 design vehicle:

Positive moment:

From Table 7-2,

$$C = M_n^+ = 317.011k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 317.011k - ft = 242.51k - ft$$

From Table 5-1,

$$D = M_{DL}^+ = 40.11k - ft$$

From Table 6-21,

$$L = M_{LL}^+ = 79.94\,k - ft$$

For the Operating Level rating:

$$RF = \frac{242.51k - ft - (1.25 \times 40.11k - ft)}{1.35 \times 79.94k - ft} = 1.78$$

For the Inventory Level rating:

$$RF = \frac{242.51k - ft - (1.25 \times 40.11k - ft)}{1.75 \times 79.94k - ft} = 1.38$$

Shear:

From Table 7-2,

$$C = V_n = 36.935k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 36.935k = 28.255k$$

From Table 5-1,

$$D = V_{DL} = 11.89k$$

From Table 6-21,

$$L = V_{LL} = 22.16k$$

For the Operating Level rating:

$$RF = \frac{28.25k - (1.25 \times 11.89k)}{1.35 \times 22.16k} = 0.45$$

For the Inventory Level rating:

$$RF = \frac{28.25k - (1.25 \times 11.89k)}{1.75 \times 22.16k} = 0.35$$

Since the HL93 load rating is less than 1 for the positive moment, it is necessary to perform the analysis using the legal load vehicles.

Using the Type 3 (Legal Load): exterior beam

Positive moment:

From Table 7-2,

$$C = M_n^+ = 317.011k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 317.011k - ft = 242.51k - ft$$

From Table 5-1,

$$D = M_{DL}^+ = 40.11k - ft$$

From Table 6-7,

$$L = M_{LL}^+ = 44.5589k - ft$$

$$RF = \frac{242.51k - ft - (1.25 \times 40.11k - ft)}{1.80 \times 44.5589k - ft} = 2.40$$

Shear:

From Table 7-2,

$$C = V_n = 36.935k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 36.935k = 28.255k$$

From Table 5-1,

$$D = V_{DL} = 11.89k$$

From Table 6-7,

$$L = V_{LL} = 16.4718k$$

$$RF = \frac{28.255k - (1.25 \times 11.89k)}{1.80 \times 16.4718k} = 0.45$$

LRFR method: interior beam

Load rating for the interior beam at the edge using the HL93 design vehicle.

Negative moment:

From Table 7-4,

$$C = M_n^- = 1166.84k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 1166.84k - ft = 892.633k - ft$$

From Table 5-2,

$$D = M_{DL}^- = 618.49k - ft$$

From Table 6-22,

$$L = M_{LL}^- = 676.93k - ft$$

For the Operating Level rating:

$$RF = \frac{892.633k - ft - (1.25 \times 618.49k - ft)}{1.35 \times 676.93k - ft} = 0.13$$

For the Inventory Level rating:

$$RF = \frac{892.633k - ft - (1.25 \times 618.49k - ft)}{1.75 \times 676.93k - ft} = 0.10$$

Shear:

From Table 7-4,

$$C = V_n = 201.902k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 201.902k = 154.46k$$

From Table 5-2,

$$D = V_{DL} = 59.26k$$

From Table 6-22,

$$L = V_{LL} = 64.56\,k$$

For the Operating Level rating:

$$RF = \frac{154.46\,k - (1.25 \times 59.26\,k)}{1.35 \times 64.56\,k} = 0.92$$

For the Inventory Level rating:

$$RF = \frac{154.46\,k - (1.25 \times 59.26\,k)}{1.75 \times 64.56\,k} = 0.71$$

Since the HL93 load rating is less than 1 for the negative moment, it is necessary to perform the analysis using the Legal Load vehicles.

Using the Type 3 (Legal Load): Interior Beam

Negative moment:

From Table 7-4,

$$C = M_n^- = 1166.84\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 1166.84k - ft = 892.633k - ft$$

From Table 5-2,

$$D = M_{DL}^{-} = 618.49k - ft$$

From Table 6-8,

$$L = M_{LL}^{-} = 341.78k - ft$$

$$RF = \frac{892.633k - ft - (1.25 \times 618.49k - ft)}{1.80 \times 341.78k - ft} = 0.19$$

Shear:

From Table 7-4,

$$C = V_n = 201.902k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 201.902k = 154.46k$$

From Table 5-2,

$$D = V_{DL} = 59.26k$$

From Table 6-8,

$$L = V_{LL} = 33.6818k$$

$$RF = \frac{154.46k - (1.25 \times 59.26k)}{1.80 \times 33.6818k} = 1.33$$

LRFR method: interior beam

Load rating for the interior beam at the mid-span using the HL93 design vehicle:

Positive moment:

From Table 7-4,

$$C = M_n^+ = 671.688\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 671.688k - ft = 513.841k - ft$$

From Table 5-2,

$$D = M_{DL}^+ = 114.34\,k - ft$$

From Table 6-22,

$$L = M_{LL}^- = 216.46\,k - ft$$

For the Operating Level rating:

$$RF = \frac{513.841\,k - ft - (1.25 \times 114.34\,k - ft)}{1.35 \times 216.46\,k - ft} = 1.27$$

For the Inventory Level rating:

$$RF = \frac{513.841\,k - ft - (1.25 \times 114.34\,k - ft)}{1.75 \times 216.46\,k - ft} = 0.98$$

Since the HL93 load rating is less than 1 for the positive moment, it is necessary to perform the analysis using the legal load vehicles.

Using the Type 3 (Legal Load): interior beam

Positive moment:

From Table 7-4,

$$C = M_n^+ = 671.688\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 671.688k - ft = 513.841k - ft$$

From Table 5-2,

$$D = M_{DL}^+ = 114.34\,k - ft$$

From Table 6-8,

$$L = M_{LL}^- = 123.998\,k - ft$$

$$RF = \frac{513.841k - ft - (1.25 \times 114.34\,k - ft)}{1.80 \times 123.998\,k - ft} = 1.66$$

LRFR method: interior beam

Load rating for the interior beam at 1/3 of the mid-span length using the HL93 design vehicle:

Negative moment:

From Table 7-4,

$$C = M_n^- = 726.178\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 726.178k - ft = 555.526k - ft$$

From Table 5-2,

$$D = M_{DL}^{-} = 288.44k - ft$$

From Table 6-22,

$$L = M_{LL}^{-} = 359.71k - ft$$

For the Operating Level rating:

$$RF = \frac{555.53k - ft - (1.25 \times 288.44k - ft)}{1.35 \times 359.71k - ft} = 0.40$$

For the Inventory Level rating:

$$RF = \frac{555.53k - ft - (1.25 \times 288.44k - ft)}{1.75 \times 359.71k - ft} = 0.31$$

Shear:

From Table 7-4,

$$C = V_n = 128.771k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 128.771k = 98.51k$$

From Table 5-2,

$$D = V_{DL} = 37.15k$$

From Table 6-22,

$$L = V_{LL} = 50.91k$$

For the Operating Level rating:

$$RF = \frac{98.51k - (1.25 \times 37.15k)}{1.35 \times 50.91k} = 0.76$$

For the Inventory Level rating:

$$RF = \frac{98.51k - (1.25 \times 37.15k)}{1.75 \times 50.91k} = 0.58$$

Since the HL93 load rating is less than 1 for the negative moment, it is necessary to perform the analysis using the legal load vehicles.

Using the Type 3 (Legal Load): interior beam

Negative moment:

From Table 7-4,

$$C = M_n^- = 726.178k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 726.178k - ft = 555.526k - ft$$

From Table 5-2,

$$D = M_{DL}^- = 288.44k - ft$$

From Table 6-8,

$$L = M_{LL}^- = 195.89k - ft$$

$$RF = \frac{555.526k - ft - (1.25 \times 288.44k - ft)}{1.80 \times 195.89k - ft} = 0.55$$

Shear:

From Table 7-4,

$$C = V_n = 128.771k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 128.771k = 98.51k$$

From Table 5-2,

$$D = V_{DL} = 37.15k$$

From Table 6-8,

$$L = V_{LL} = 28.3521k$$

$$RF = \frac{98.15k - (1.25 \times 37.15k)}{1.80 \times 28.3521k} = 1..02$$

LRFR method: interior beam

Load rating for the interior beam at 2/3 of the mid-span length using the HL93 design vehicle:

Positive moment:

From Table 7-4,

$$C = M_n^+ = 918.522k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 918.552k - ft = 702.67k - ft$$

From Table 5-2,

$$D = M_{DL}^+ = 66.81k - ft$$

From Table 6-22,

$$L = M_{LL}^+ = 135.55k - ft$$

For the Operating Level rating:

$$RF = \frac{702.67\,k-ft - (1.25 \times 66.81\,k-ft)}{1.35 \times 135.55\,k-ft} = 3.38$$

For the Inventory Level rating:

$$RF = \frac{702.67\,k-ft - (1.25 \times 66.81\,k-ft)}{1.75 \times 135.55\,k-ft} = 2.61$$

Shear:

From Table 7-4,

$$C = V_n = 75.911k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 75.911k = 58.07k$$

From Table 5-2,

$$D = V_{DL} = 20.22\,k$$

From Table 6-22,

$$L = V_{LL} = 37.59\,k$$

For the Operating Level rating:

$$RF = \frac{58.07\,k - (1.25 \times 20.22\,k)}{1.35 \times 37.59\,k} = 0.65$$

For the Inventory Level rating:

$$RF = \frac{58.07\,k - (1.25 \times 20.22\,k)}{1.75 \times 37.59\,k} = 0.50$$

Since the HL-93 load rating is less than 1 for the positive moment, it is necessary to perform the analysis using the Legal Load vehicles.

Using the Type 3 (Legal Load): interior beam

Positive moment:

From Table 7-4,

$$C = M_n^+ = 918.522\,k - ft$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times M_n = 0.85 \times 1.0 \times 0.9 \times 918.552k - ft = 702.67k - ft$$

From Table 5-2,

$$D = M_{DL}^+ = 66.81k - ft$$

From Table 6-8,

$$L = M_{LL}^+ = 75.5564\,k - ft$$

$$RF = \frac{702.67\,k - ft - (1.25 \times 66.81k - ft)}{1.80 \times 75.5564\,k - ft} = 4.55$$

Shear:

From Table 7-4,

$$C = V_n = 75.911k$$

Applying the conditions and reduction factors

$$C = \phi_c \phi_s \phi \times V_n = 0.85 \times 1.0 \times 0.9 \times 75.911k = 58.07k$$

From Table 5-2,

$$D = V_{DL} = 20.22\,k$$

From Table 6-8,

$$L = V_{LL} = 22.8436\,k$$

$$RF = \frac{58.07\,k - (1.25 \times 20.22\,k)}{1.80 \times 22.8436\,k} = 0.80$$

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Load Rating of the Lahontan Arch Spillway Bridge at Fallon, NV

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Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT

This study focuses on the load-rating analysis of a five-span, concrete arch bridge located at Fallon, NV. The Lahontan Dam Spillway Bridge was constructed in 1915 and, after approximately 94 years of service, the bridge has deteriorated. The U.S. Army Engineer District, Sacramento, inspected the bridge in 2009 and determined that the bridge is in poor condition. Advanced section loss, deterioration at different locations along the full length of the bridge, and severe spalling in the three main arch beams and in the piers were the main defects observed. These defects put in jeopardy the structural integrity of the bridge; thus, the bridge is currently closed to traffic. The purpose of the load rating was to identify the safe load capacity of the bridge under today's design vehicle and legal loads. The load ratings were performed with two specifications, the American Association of State Highway and Transportation Officials' (AASHTO) *Manual for the Condition Evaluation of Bridges* load factor rating and the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*. The load ratings were performed for the following types of loading: notional HS20-44; the HL93; and the legal load for vehicles Type 3, Type 3S2, and Type 3-3. The rating factors' results confirmed the poor condition of the bridge described in the 2009 inspection report. It was concluded that the bridge design does not follow current standards; thus, a weight limit sign is required to be posted on the bridge. The recommended weight limit based on the LFR at inventory level is 7 tons of the Type 3 loading, 11 tons of the Type 3S2 loading, and 14 tons of the Type 3-3 loading.

15. SUBJECT TERMS

	LFR	Moving load
Arch beam	Load rating	Reinforced concrette
FEM	LRFR	SAP2000

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